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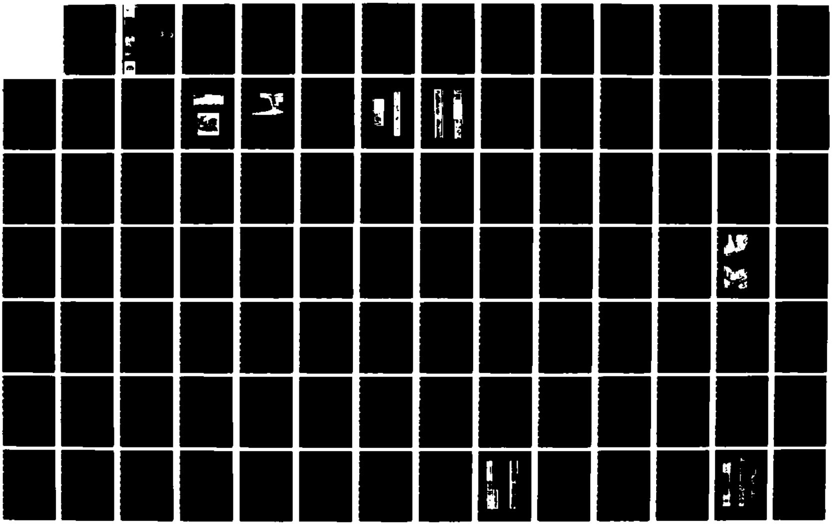

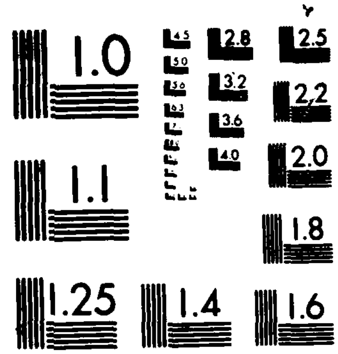

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CONDITION SURVEY OF LOCK NO. 7, MONONGAHELA RIVER

by

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Structures Laboratory

DEPARTMENT OF THE ARMY
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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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A condition survey was performed at Lock and Dam No. 7 on the Monongahela River in Pennsylvania to determine the quality of concrete, the extent of possible concrete damage, the processes causing distress of the concrete, and selected physical and mechanical properties of the concrete and foundation materials. The field investigation included a visual examination of the lock and drilling operations to recover concrete and foundation core. Results of the field investigation and laboratory tests indicated that the processes causing distress in the concrete are freezing and thawing action and alkali-silica reaction. The concrete is extensively damaged on and near the top of the guide, guard, and lock walls. Overall the lock concrete is in poor condition evidenced by fine to wide cracking, light to severe scaling, and large spalls. Low quality concrete exists to depths of 2-ft vertically and 0.9-ft horizontally in the guide, guard, and lock walls. Concrete beneath the damaged zones is considered to be of good quality.

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PREFACE

The work described in this report was performed for the US Army Engineer District, Pittsburgh, by personnel of the US Army Engineer Waterways Experiment Station (WES). The work was authorized by DA Form 2544, No. ORPED-82-69, dated 12 August 1982.

The testing program was accomplished under the direction of Mr. Bryant Mather, Chief, Structures Laboratory (SL), WES, and Mr. John M. Scanlon, Jr., Chief, Concrete Technology Division (CTD), SL. The majority of the core drilling was conducted by the US Army Engineer District, Mobile, under the direction of Mr. Pat Douglas. Some horizontal drilling was conducted by a private drilling company from Pittsburgh and was under the direction of Mr. Frank Pehr, Geotechnical Branch, Pittsburgh District. Laboratory work in the CTD was done with the assistance of Mr. F. S. Stewart, Mrs. Joyce C. Ahlvin, and Mr. G. Sam Wong. Mr. Richard L. Stowe was Project Leader for the investigation. Mr. Stowe prepared this report.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

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* On file in the Pittsburgh District.

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
microns	1	micrometres
pounds (force) per square inch	0.006894757	megapascals
pounds (force)	4.448222	newtons
pounds (force) per foot	14.59390	newtons per metre
pounds (mass) per cubic foot	16.018463	kilograms per cubic metre
tons (force) per square foot	95.76052	kilopascals

CONDITION SURVEY OF LOCK NO. 7, MONONGAHELA RIVER

PART I: INTRODUCTION

Project Description

1. The following general description is taken from the third periodic inspection report of Lock and Dam No. 7 (U. S. Army Engineer District, Pittsburgh 1979).

> Lock and Dam 7, Monongahela River, was constructed in 1923-26. The 56 foot by 360 foot lock chamber is located on the left bank. The lock walls and gate sills are unreinforced concrete gravity sections founded on rock. The fixed crest dam consists of an unreinforced concrete section founded on rock. The dam extends 610 feet from the river lock wall to the concrete abutment on the right bank and has a normal lift of 15 feet. The project has been operated and maintained since 11 November 1925. The lock walls were resurfaced by the District's repair party, working from 10 June 1940 to 19 September 1940.

A general plan view of the project site is presented in Plate 1; this plate is taken from the first periodic inspection report (U. S. Army Engineer District, Pittsburgh 1972).

Background

2. In August of 1982, the Waterways Experiment Station (WES) was requested by the U. S. Army Engineer District, Pittsburgh (ORP for Ohio River, Pittsburgh), to submit a proposal for a condition survey of Locks 7 and 8 on the Monongahela River (Mon River). Engineering information and data were supplied by the District for review (see Table 1). The materials included design, construction, and operational documents, and a project photograph. The information and data were used in working up the WES proposal and were used throughout our work. The foundation

drilling program and the majority of the testing program was established by the District.

Objectives

3. The objectives of the condition survey are: (a) identify the processes or materials causing distress or failure of the concrete and the probable extent of such damage, (b) determine the ability of the concrete to perform satisfactorily under anticipated conditions of future service, and (c) determine selected physical and mechanical properties of the foundation. The Pittsburgh District is scheduled to perform structural stability analyses.

Scope

4. This report presents the findings of an inspection of the project prior to drilling. The drilling effort involved in recovering samples of concrete and foundation work is discussed. The physical condition and extent of damage of in-place concrete are described using visual, petrographic, and physical property information and data. Selected physical properties of core samples were determined using standard Corps of Engineers test methods.

PART II: PRELIMINARY STUDY

5. The author and Mr. Frank Pehr of the District office made an inspection of Lock No. 7. The purpose was to ascertain the general condition of the lock, guide, and guard walls, and determine the location of borings for obtaining concrete and foundation cores. Observed concrete deficiencies are discussed in the following paragraphs. These observations generally agree with those observations made by others as presented in the periodic inspection reports (U. S. Army Engineer District, Pittsburgh 1972, 1974, and 1979). Portions or all of the top surface of the lock, guide, and guard walls were resurfaced in the early 1940's. The condition of the concrete below these resurfaced portions was evaluated with core borings.

Upper Guide Wall

6. The top surface of the upper guide wall concrete appears to be in fair to good condition. There are a few small areas of light to medium scaling, a few small spalls, medium to large popouts, a few longitudinal cracks (both fine and wide)* upwards of 70 ft long, and transverse cracks (fine to wide) averaging one every 8 ft for the upper one-third of the wall. The transverse cracks are less frequent for the last two-thirds of the wall. Small areas of random and pattern cracks filled with efflorescence and local areas with alkali-aggregate reaction rims are present.

7. The vertical face of the wall concrete is in poor condition; it is badly gouged, contains small to large spalls and local severe scaled areas. One to two vertical cracks (wide) are present in each monolith. There are some large joint spalls at the monolith joints.

8. Due to the high acid content of the water, which prevailed until the early 1960's, surface fines near and below the water level

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* Crack widths: fine - generally less than 1 mm; medium - between 1 and 2 mm; wide - over 2 mm. See American Concrete Institute 1980 for definitions of terms associated with the durability of concrete.

have been removed from the concrete leaving an architectural exposed aggregate appearance.

9. New concrete and mortar patches on the top of the wall surface are generally in fair to good condition; however, some patches are drummy. Appendix A presents typical photographs (1 through 5) of the upper guide wall surfaces.

Lower Guide Wall

10. The top surface of the lower guide wall concrete appears to be in good condition; it has light scaling and a few longitudinal and transverse cracks (fine to wide) are present. One transverse crack crosses the top of the wall and runs down the back side of the wall for about 4 ft; it is open 1/8 to 1/4 in.

11. The presence of the tailrace for the turbine discharge kept the author from inspecting the back side of the lower guide wall.

12. The vertical face of the lower guide wall concrete (chamber side) is in poor to fair condition. The first four monoliths have local areas that are lightly scaled and spalled. The remaining monoliths have medium to severe scaling and large spalled areas. The vertical monolith joints are spalled up to 10 in.; most are spalled for their full exposed length. A few horizontal and diagonal cracks are present in the wall face with efflorescence tracing and being deposited below the cracks. Appendix A presents typical photographs (6 and 7) of the lower guide wall surfaces.

Upper Guard Wall

13. The top surface of the upper guard wall concrete appears to be in fair to poor condition. Light to medium scaling and local large spalls are present. Pattern and random cracking is present with cracks being sealed or filled with debris. A few transverse cracks (wide) were observed. Extensive patching is evident with many patches drummy or broken.

14. The vertical face of the wall concrete is in fair to poor condition. Light to severe scaling and small to large spalls are present in local areas. Horizontal cracks (wide) on the river side of the wall are present near the top of the wall. Erosion near the waterline is also present. Small joint spalls are present on most exposed vertical monolith joints.

15. The upper guard wall extension cell concrete caps appear to be in good condition. The steel sheet piling is in good condition.

Lower Guard Wall

16. The top surface of the wall concrete is in fair to poor condition. Light to medium scaling and local small to large spall areas are present along with random cracking (fine to wide); the cracks are sealed or filled with debris. A few transverse cracks (wide) were observed. The upstream one-half of the wall is heavily patched. The condition of the patches ranges from poor to good. The downstream one-half of the wall contains fewer concrete deficiencies.

17. Approximately 60 percent of the vertical face of the wall concrete (chamber side) is in very poor condition; the remainder of the face is in fair to poor condition. Large areas are deeply spalled and very severely scaled. Erosion by barge and ice is evident. Vertical and horizontal cracks (wide) exist with efflorescence deposited on or adjacent to most cracks. Monolith joints are spalled their full exposed length with some spall zones measuring 12 to 14 in. deep. Monoliths No. R-18 and R-19 are more severely deteriorated than the other monoliths.

18. The top of the wall has been resurfaced and the vertical wall portion of this resurfaced concrete appears to be in fair to good condition. Leaching from directly below this resurfacing is manifested by efflorescence, thus indicating that ponded water on the top surface of the wall is filtering through cracks in the cap.

19. The vertical face of the wall concrete (river side) is in fair to poor condition. The narrow downstream section has the least concrete damage; the two horizontal surfaces of the wall (el 776 and 766)* are

*All elevations (el) cited herein are in feet referred to mean sea level.

eroded several inches and the corners are rounded. The thicker wall section exhibits all the concrete deficiencies described for the chamber side of the wall. Most of the damaged concrete is within the top 15 ft of the face.

20. Appendix A presents typical photographs (8 through 16) of the lower guard wall surfaces.

Land Lock Wall

21. The top surface of the land lock wall concrete appears to be in poor condition. The top surface of the wall has been resurfaced. There are numerous areas of light to medium scaling and local areas of small spalls. Local pattern and random cracking (wide) areas exist. The top surface concrete is extensively patched. Some patches are sound while many are drummy, spalled, and cracked. Monolith joints are spalled and, where not covered with patches, are open or partially scaled.

22. The vertical face of the land lock wall concrete is in poor condition. The concrete above upper pool elevation contains local large spalls, local small to medium scaled areas, and horizontal and vertical cracks (wide); the concrete is also moderately eroded and worn due to barge and ice impacts. Between upper and lower pool elevations, the entire surface is almost uniformly scaled (severe to very severe). As mentioned earlier, acid water attack has resulted in removal of fines leaving an architectural exposed aggregate appearance; this action contributed to the erosion of the wall concrete. Horizontal and vertical construction joints are spalled up to about 6 in. deep. Other local areas of spalling are present. Shotcrete patches appear in good condition. Most all monoliths contain one or more vertical cracks (wide) from 6 to 15 ft long. The concrete adjacent to and in the ladderways and the line hook castings is severely spalled.

23. Presented in Appendix A are photographs (17 through 22) showing typical concrete condition for the land lock wall surfaces.

River Lock Wall

24. The top surface of the river lock wall concrete appears to be in poor condition. The top surface of the wall has been resurfaced. A few local areas of small and large spalls are present as are local areas of random cracking (fine to medium). The resurfaced concrete is extensively patched with some patches sound and some drummy. Light scaling is present over much of the top surface.

25. The vertical faces (chamber and river sides) of the river lock wall concrete appear to be in poor condition. The concrete above upper pool elevation contains local large spalls, light to severe scaled areas, and horizontal and short vertical cracks (wide). Erosion and wear from barge and moving ice are evident on the chamber side. Heavy leaching, manifested by efflorescence, is occurring on both sides of the wall from just beneath the resurfaced cap and from numerous horizontal cracks (wide). The leaching indicates that rainwater is infiltrating through cracks and spall areas in the top surface of the wall and exiting on the wall face. The concrete between upper and lower pool elevations contains those concrete deficiencies that were described for the land lock wall. These deficiences are at an advanced stage of deterioration in the river lock wall. For example, the spalled areas are larger and some of the gunite patches are missing. The monolith joints are spalled their full exposed length with local spalls measuring 12 in. deep.

26. The galleries in the lock walls were not inspected. The description from the second periodic inspection report depicts the condition of the concrete in the galleries (U. S. Army Engineer District, Pittsburgh 1974).

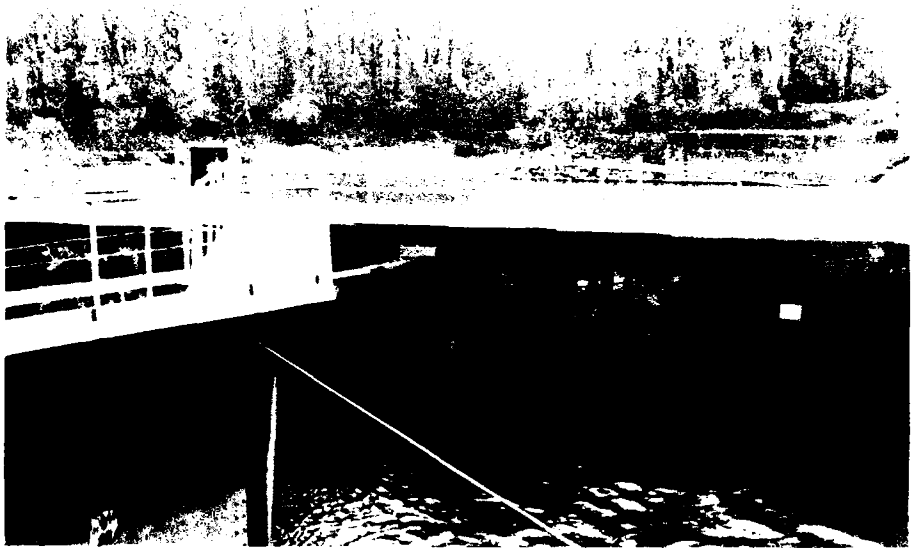
> The pipe galleries in both lock walls contain numerous cracks. Some of the cracks in the ceiling are leaching. In both galleries, a few longitudinal cracks in the ceiling extend up into the overhead stairwells. The floor, walls and ceiling of each gallery contain several small areas of spalling. The most serious and extensive cracks are located in the downstream end of the river wall gallery. The end monolith is segmented by several structural cracks, one of which has opened approximately 1 inch.

27. Photograph 14 in Appendix A illustrates a diagonal structural crack on the river side of the lock wall. The crack is probably an external expression of one of the structural cracks observed from inside the river wall gallery. Photograph 27 in Appendix A illustrates a similar structural crack on the river side of the river lock wall in the area of the upper miter gate recess. It was not within the scope of this investigation to determine the possible cause of the structural cracks.

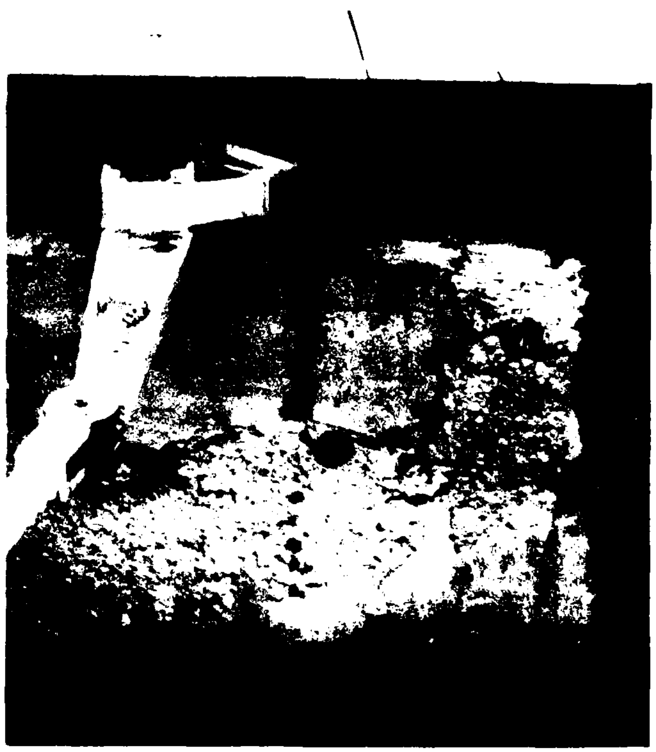
28. Appendix A presents typical photographs (23 through 27) of the river lock wall surfaces.

Miter Gate Recesses

29. The surface concrete in the miter gate recesses appears to be in poor condition. Large spalls and light to medium scaling are present. The concrete around the gate machinery looks like resurfaced concrete; it is in general lightly damaged and most corners are square. The extent of damaged concrete from a visual standpoint is not as great in the gate recesses as observed at Lock No. 8 Monongahela River. Figure 1 illustrates typical concrete damage in the gate recesses.



a. Lower miter gate recess, river wall



b. Close-up of machinery area shown in 1.a. above

Figure 1. Concrete surface damage in miter gate recess
(continued)

c. Lower miter gate recess, land lock wall, showing spalling, scaling, and some evidence of leaching

Figure 1. (Concluded)

PART III: DRILLING OPERATION

30. At the completion of the inspection of the lock, boring locations were assigned in areas that were representative of the various degrees of deteriorated concrete; see Plate 2 for boring locations. Guidance was given by District personnel on the total number of borings. This guidance was partially based upon the time remaining before the severe winter weather set in. Pertinent boring information concerning the boring number, the monolith number the boring was placed in, depth, core size, boring direction, elevation top of boring, elevation top of rock, and elevation bottom of boring is presented in Table 2.

31. Total footage drilled was 310.6 ft of concrete and 95.4 ft of foundation rock; total number borings drilled was 26. The bedrock core from borings NG WES L-1-82 and R-1-82* was preserved for possible laboratory testing. Due to a misunderstanding in communications, the bedrock core from boring R-3-82 was not preserved. No laboratory test samples were obtained from the bedrock portion of this boring. Procedures for preserving and handling the bedrock are given in Test Standard RTH 103-80 (U. S. Army Engineer Waterways Experiment Station,1980). Color photographs of the core from the vertical borings are presented in Exhibit A.

32. Core recovery was good in all borings; the average core recovery was 99 percent. The general condition of the concrete and foundation core is illustrated in Figures 2 and 3.

33. Drilling equipment consisted of a Failing 43-5A skid rig and a portable electric drill for taking horizontal core. A Diamond Core Drill Manufacturers Association standard 6-in. by 7-3/4-in. and 4-in. by 5-1/2-in. double tube swivel tube core barrel was used with diamond bits to obtain the concrete and bedrock core in the vertical borings. A single tube core barrel with a diamond bit was used to take the horizontal core. Access to the drill holes on top of the lock walls was by crane

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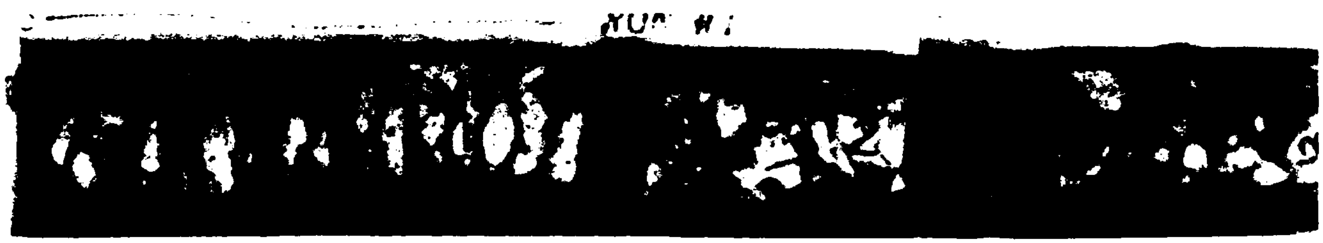
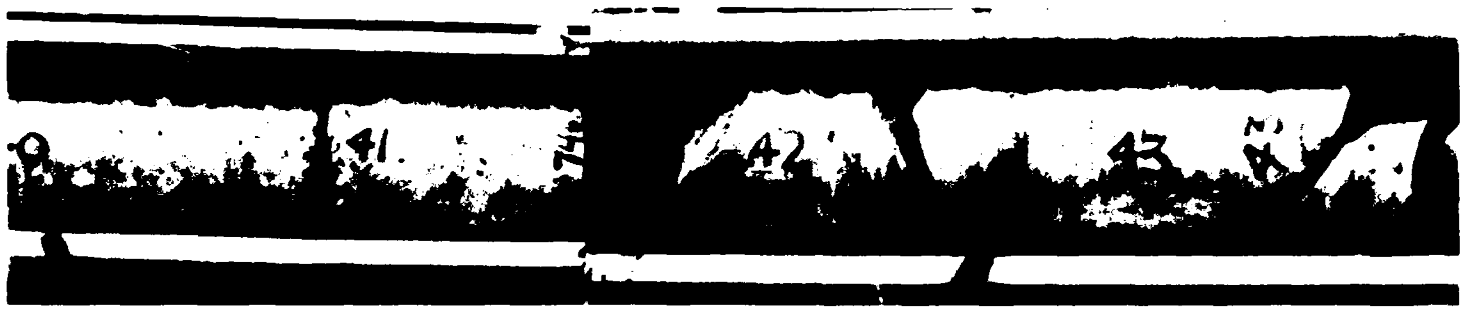
* NG = New Geneva; WES = drilling agency, Waterways Experiment Station; L = lock wall; R = River wall; number after L indicates number of boring; and 82 is year boring was made (1982). Most horizontal borings are designated NG PDD etc.; the PDD = Pittsburgh Industrial Diamond Products.

a. Vertical concrete core, boring L-1-82, land lock wall, upper miter gate monolith, damage due to freezing and thawing action; damage 0.0 to 1.0 ft



b. Vertical concrete core, boring L-4-82, lower guide wall. Note subparallel cracking due to freezing and thawing, damage from 0.0 to 1.6 ft

Figure 2. Typical damaged concrete core



Concrete

Foundation Rock

a. Concrete/foundation rock contact (loose bond), boring R-1-82 in river lock wall upper miter gate monolith block. Depth 40.0 to 43.6 ft

b. Foundation rock, boring R-1-82, depth 43.6 ft to 46.9 ft

Figure 3. Intact and broken foundation rock core

and by a cable hoist. Access to the horizontal boring locations was by a marine work platform supplied by the lockmaster.

34. Personnel from the U. S. Army Engineer District, Mobile, drilled the vertical borings and one of the horizontal borings. The Pittsburgh District contracted with the Pitt Industrial Diamond Products, Inc. to drill the remaining horizontal borings at Lock 7. This was done to expedite the drilling operation.

PART IV: GEOLOGY

35. The description of the site geology is taken from the first periodic inspection report (U.S. Army Engineer District, Pittsburgh 1972)

> The Monongahela River in the vicinity of Greensboro, Pennsylvania, and Lock and Dam 7 flows in a series of entrenched meanders through the maturely dissected Kanawha section of the Appalachian Plateaus province in a valley carved in the Allegheny, Conemaugh, and Monongahela formations of the Pennsylvanian system. Regional relief varies from a minimum elevation of approximately 740 feet in the bed of the Monongahela River to a maximum elevation of 1263 feet in the uplands. The valley width is approximately 1200 feet with the river comprising 700 feet and flood plain and valley fill deposits at 500 feet.
>
> The rocks of the region consist of flat lying cyclic sediments; chiefly indurated clays, siltstones, sandstones, and coal, all of which are members of the Middle Conemaugh formation. The site of Lock and Dam 7 lies between the Lambert Syncline on the west and the Fayette Anticline on the east.
>
> The overburden at the site of Lock and Dam 7, Monongahela River, is comprised mainly of alluvial soils in the river bottom with alluvial deposits comprising the flood plain on the abutment side and colluvial deposits comprising the lock side. The alluvial materials consists of silty sandy gravels being either brown or gray in color. The colluvial materials consist of reddish grey silty clays with numerous rock fragments.

36. A geologic profile sheet was not drawn up due to the limited number of borings taken into the foundation. The three field logs presented in Appendix B (L-1-82, R-1-82, and R-3-82) indicate relatively flat laying beds and continuity of lithologic units between the three borings. There are no apparent indications of any structural offsets. The concrete to foundation rock bond was tight in R-3-82 and loose in R-1-82 and L-1-82.

37. The foundation material (3 to 6 ft) directly below the concrete lock walls was, in general, similar in all three cores. The rock is a moderately hard, fine-grained sedimentary rock that will be referred to in this report as a shale; it was very nearly classified as indurated

clay. It is thin bedded and slakes upon drying. The rock cores were fractured and badly broken to the point that no samples were obtained for testing. Probable reasons for the fractured and broken shale core are: (a) blasting during the original construction which fractured the in situ rock and (b) drilling action and core removal from the drill barrel of the already fractured rock. If this rock unit is again drilled for test samples, it is suggested that different or more appropriate drilling equipment or core retrieving devices be used to increase the likelihood of recovering suitable test specimens. The average thickness beneath the concrete is 4.8 ft.

38. The next approximately 6 ft of rock is coarser grained than the rock above. The rock represents a transitional zone from the shale above to the sandstone below. It contains interbedded shale, siltstone, and sandstone. Borings R-1-82 and R-3-82 contain this interbedded rock. Within this 6-ft depth in boring L-1-82, a limey rock exists that is classified as a limestone. It is one of those "hard to call rock units" because it could be classified as a calcareous shale, calcareous siltstone, or calcareous sandstone. The limestone probably represents a facies change. The average thickness of this unit is 6.7 ft.

39. The next approximately 12 ft of rock is a competent hard sandstone bed containing a few thin clayey seams at 62.5-ft depth in boring R-1-82 and at 62.7-ft depth in boring R-3-82. The sandstone is generally medium grained.

40. Below the sandstone there is a moderately hard to soft medium gray indurated clay. The rock is similar in all three borings. The thickness of this unit was 10 ft in the one boring that went through it.

41. Boring R-3-82 was the only one taken deep enough to recover rock below the indurated clay. A thin sandstone unit and interbedded sandstone and shale were identified in this deeper interval.

PART V: TEST SPECIMENS AND TEST PROCEDURES

42. Field procedures for preparing the foundation rock for testing were as follows. After removal from the core barrel the core was marked to indicate boring location and depth. Photographs were taken and a quick drilling log prepared. Moistureproofing was accomplished by waxing the core. The core was wrapped in thin polyethylene (Saran Wrap), wrapped with cheese cloth and then coated with a lukewarm wax mixture to an approximate 1/4-in. thickness. The wax consisted of a 1 to 1 mixture of paraffin and microcrystalline wax. The core was placed in a wooden cone box and cushioned with sawdust. RTH 103-80 was used as guidance (U. S. Army Engineer Waterways Experiment Station 1980).

Cores Received

43. Concrete and foundation rock samples from the 26 borings were shipped by government motor freight. All samples were received in good condition; no sample breakage was detected. Pertinent core information is presented in Table 3.

Selection of Test Specimens

44. A detailed visual examination of all core was made in the laboratory to supplement the field boring logs and to assist in the selection of representative test specimens. Concrete specimens were selected for testing based upon physical condition and depth in order to obtain representative properties through the structure. In the three vertical borings through the lock walls specimens were taken from the top, middle, and bottom of the core.

45. As a rule, test specimens are selected to be representative of the bedrock in close proximity to the base of the structure. As mentioned in Part IV, the rock core recovered from the shale unit beneath the lock wall concrete was fractured to the point that intact test specimens could not be obtained. After the petrographic examination was

completed, it was agreed that the indurated clay, some 18 ft below the shale, was quite similar to the shale; both rocks were slightly limey, both slaked, both contained fine-grained material, and both rocks had a similar clayey feel. Because of these similarities and the fact that the shale could not be tested, the indurated clay was tested in direct shear.

46. It is recognized that the shear strengths obtained from the indurated clay will likely be different than the shear strengths of the shale. However, it is reasonable to assume that the difference will not be great based upon the physical similarities of the two rocks. Until the shale can be resampled and tested in direct shear, it is suggested that the peak and residual shear strengths of the indurated clay be used for the shale unit for purposes of sliding stability analysis.

47. Test specimens were selected for testing concurrent with the detailed logging of core; the logging began 3 weeks after the core arrived at the laboratory. The test specimens were rewrapped and stored in a moist curing room until time for testing; the moist room is maintained at 73.4 $\pm$ 3 F (23 $\pm$ 1.7 C). The test assignment locations can be obtained from appropriate tables of test results.

Laboratory Test Program

Concrete Cores

48. The testing program of the concrete cores consisted of the following tests and examination.

- a. Petrographic examination.
- b. Unit weight, γ .
- c. Compression wave velocity, V_p .
- d. Compressive strength.
- e. Elastic modulus, E .
- f. Poisson's ratio, ν .

Rock Cores

49. The testing of the bedrock cores consisted of the following tests. The tests are grouped under either characterization tests or engineering design tests.

a. Characterization tests.

(1) Effective (as-received) unit weight, γ_m .

(2) Water content, w .

(3) Compressive strength, q_u .

b. Engineering design tests.

Direct shear strength.

(a) Concrete on rock, precut (residual).

(b) Intact (peak and residual).

(c) Rock on rock, precut (residual).

Test Procedures

50. The characterization properties tests and the engineering design properties tests were conducted in accordance with the appropriate test method tabulated below:

Property	Test Method Rock	Test Method Concrete
Characterization		
Effective Unit Weight (As-Received), γ_m	RTM 109-80	CRD-C23-84
Water Content, w	RTM 106-80	RTM 106-80
Compressional Wave Velocity, V_p	RTM 110-80	CRD-C51-70
Compressive Strength, q_u	RTM 111-80	CRD-C14-85
Engineering Design		
Elastic Modulus, E	RTM 201-80	CRD-C19-83
Poisson's Ratio, ν	RTM 201-80	CRD-C19-83
Direct Shear Strength	RTM 203-80	
Petrographic Examination	RTM 109-80*	CRD-C57-78

* The applicable portions of the test procedures used in conducting the petrographic examination of the rock and concrete are described in Appendixes C and D, respectively.

51. For the compression test, the specimens were cut with a diamond-blade saw and the cut surfaces ground flat to 0.001 in.; specimens were checked for planeness, parallel ends, and the perpendicularity of ends to the axis of the specimen. Electrical resistance strain-gages and linear variable differential transducers were used for strain measurements. The modulus of elasticity and Poisson's ratio were computed from the strain measurements. Axial specimen load was applied using a 440,000-lbf capacity universal testing machine.

Pullout Resistance

52. Hard sandstone cores with nominal diameters of 6 in. were placed upright in 30-in.-diameter molds. A concrete mixture was placed into the mold embedding the core to its full length. The concrete portion served two purposes. First, it acted as a resistance block allowing the reinforcing bar to be pulled, and second, it served as a host material in case the core instead of the reinforcing bar was pulled out.

53. After the concrete had cured and attained an approximate strength of 1,000 psi, a 1-in.-diameter hole was drilled in the center of the sandstone cores. A thin-wall diamond bit was used resulting in a smooth-walled borehole. A No. 4 reinforcing bar was grouted the full depth of the core using a commercially available premix grout for anchoring bolts and dowels. After the grout attained a 5-day strength of 3550 psi, the bars were pulled using the setup depicted in Figure 4. Concrete strength at test time (17 days) was 3450 psi. Total weight of the suspended specimen was taken into account in calculating the bond stress.

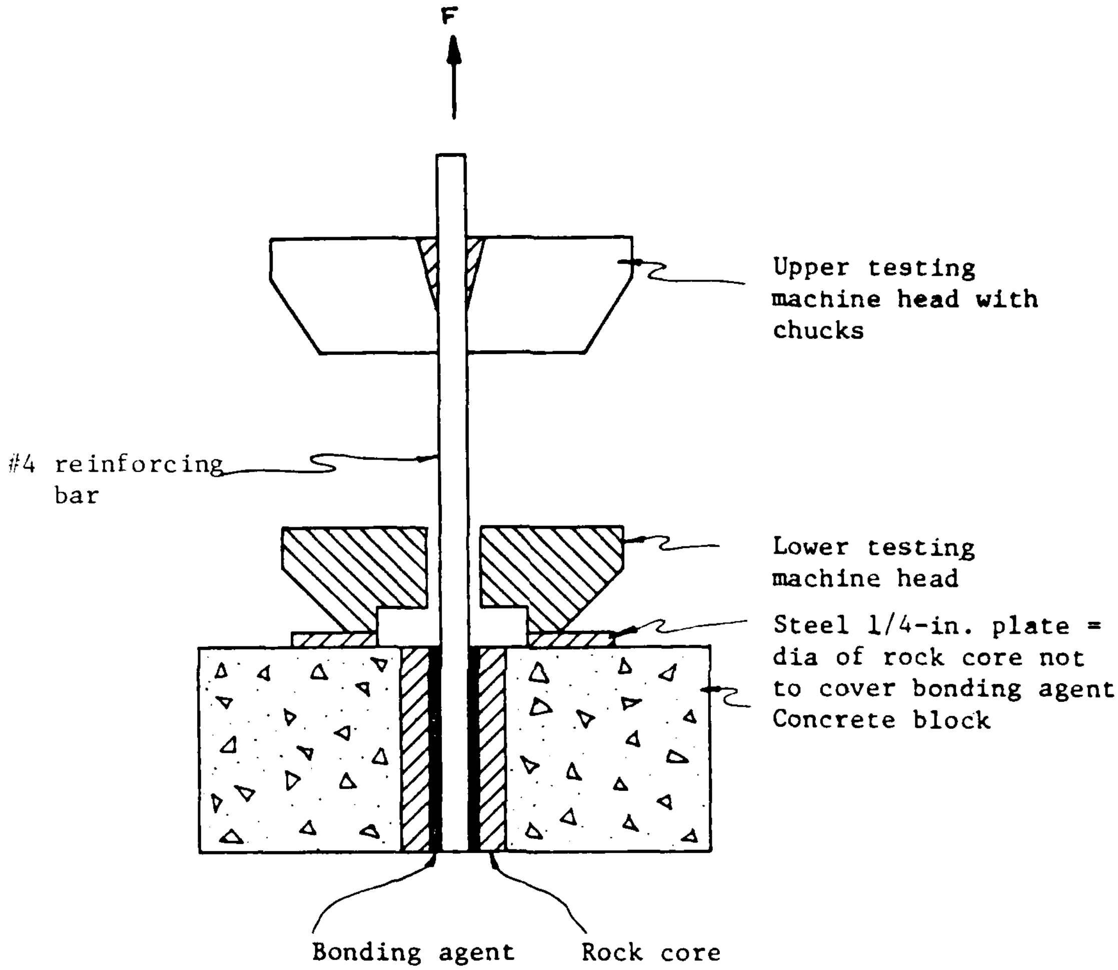


Figure 4. Section showing test configuration for the reinforcing bar pullout tests

PART VI: TEST RESULTS AND DISCUSSION

Petrographic Examination

54. The petrographic examination accomplished the following: (a) classified the foundation rock core, (b) identified the aggregates in the concrete overlays and in the original concrete, (c) identified the processes causing distress and failure in the concrete, and (d) determined the extent of concrete damage in all cores. The classification of the foundation rock has been presented in Part IV: Geology; details of the examination of the rock are presented in Appendix C. Pieces of concrete core from 9 of the 26 borings were examined; see Appendix D for detailed results.

Concrete Aggregates

55. Vertical cores in the lock wall indicate areas with and without overlay concrete. The newer concrete is air entrained and contains 1-in. maximum size natural gravel. The gravel is a well rounded to subangular calcareous rock.

56. The original concrete is not air entrained. It is composed of 2-1/2-in. nominal maximum size natural gravel and natural sand. The gravel consists predominantly of sandstone and chert with minor amounts of limestone and coal. This composition is common for gravels and sands from the region. The concrete is well consolidated and quite similar throughout the lock structure.

Cause of Concrete Damage

57. The overlay concrete is in generally good condition; some air voids are lined with varying amounts of alkali-silica reaction products.

58. The major processes causing concrete in the lock structure to crack and break up are cycles of freezing and thawing and alkali-silica reaction. It was not within the scope of this investigation to determine

which one of these processes occurred first, or which process has caused the most damage to the concrete.

59. Evidence of freezing and thawing is manifested in the cores by closely spaced parallel cracks; such cracking occurred in over one-half of the cores. Scaling of the mortar and spalling of the concrete were also observed on the exposed ends of the cores. Evidence of alkali-silica reaction was observed in the form of efflorescence, pattern and random cracking, expansion, chalky crack surfaces, and rings around aggregates. Some white reaction material occurred on most cracked core surfaces, especially in areas where broken and rubble concrete was found.

Extent of Damage

60. The extent of the concrete damage observed in the cores during the petrographic examination is included under the subheading Concrete Quality. By doing so, the extent of concrete damage seen in all the cores will be discussed at one time.

Reinforcing Bar Pullout Resistance

61. The pullout resistance tests were conducted on cores recovered from the sandstone bed encountered in the borings. Cores from between elevations 732.2 and 729.7 were selected. The top surface of the sandstone bed is an average 12 ft below the base of the concrete lock walls; the average thickness of the sandstone bed is 12 ft. The setup for the rebar pullout test is sketched in Figure 4.

62. The tabulations below present the failure mode and the pullout loads of the test specimens.

Failure Mode of Pullout Specimens

Specimen No.	Failure Mode
L-1-82, 57.8-58.8	Grout yielded
L-1-82, 56.4-57.4	Reinforcing bar failed
L-1-82, 55.3-56.3	Reinforcing bar failed

Pullout Loads of Test Specimens

Specimen No.	Core Diameter, in.	Hole Diameter, in.	Hole Length, in.	Surface Area, in.^{2}	Pullout Strength Total Load, lbf	Pullout Strength Pounds per foot
L-1-82, 57.8-58.8	6.00	1.0	12.00	37.70	14,840	15,000
L-1-82, 56.4-57.4	5.55	1.0	11.95	37.54	15,200	15,000
L-1-82, 55.3-56.3	6.00	1.0	12.05	37.86	15,160	15,000

63. Using a minimum pullout resistance of 15,000 lb/ft and a safety factor of 2, an allowable of 7500 lb/ft is obtained for the specimens tested. The pullout resistance of the specimen from a depth of 57.8 to 58.8 ft was governed by the grout. However, the total load of 14,840 lb for this specimen is similar to the reinforcing bar yield load recorded from the other two tests, about 15,000 lb. For practical purposes, the minimum pullout resistance was governed by the rebar failure of the three specimens tested. The pullout resistance of the hard sandstone is:

$$\frac{7500\ \text{lb/ft}}{A} \approx \frac{7500\ \text{lb/ft}}{37.7\ \text{sq in.}} = 200\ \text{psi}$$

where A is the area between the grout and rock in a 1-in.-diameter hole 1 ft long.

Peak and Residual Shear Strength

64. A limited number of specimens of the weaker foundation rock (indurated clay) were available for testing. In addition to the indurated clay, specimens of the interbedded sandstone and shale were tested to determine if this unit had lower shear strengths than the indurated clay.

65. The direct shear test schedule originally included specimens of concrete bonded to rock. However, due to the shortage of suitable test specimens these scheduled tests were not run. Should additional

cores of the shale unit be taken, it is suggested that these tests be conducted.

66. A summary of the direct shear test results of foundation core is presented in Table 4. For the indurated clay, six intact, three precut concrete on rock, and three precut rock on rock specimens were run in the WES direct shear device. Three intact specimens of the interbedded sandstone and shale were also tested in direct shear. Individual test results, wet density, and moisture contents are presented in Plates 3-6. The shear stress versus shear deformation and the normal versus shear deformation curves are presented in Plates 7-36. The shear stress/normal stress values obtained on the indurated clay are presented in Figure 5.

67. Figure 5 illustrates that for the intact specimens there is little scatter in both the peak and residual shear stress for a given normal stress. The average peak shear strength, ϕ , angle of 39 degrees and cohesion, c , of 6.4 tsf is reasonable for the soft to moderately hard indurated clay. These strength values correlate well with similar values obtained on the indurated clay recovered at the Lock and Dam No. 8 structure (ϕ = 32 degrees and c = 4.0 tsf).*

68. The average residual shear angle, ϕ_r , for the intact indurated clay is 19 degrees and is considered reasonable for the indurated clay. The average residual shear angle for indurated clay at Lock and Dam No. 8 Monongahela River is 15 degrees. A cohesion value of 0.7 tsf was obtained during the residual tests. The sheared surfaces contained a few small asperities about 1/8 in. high and inclined about 2 to 4 degrees. This surface roughness plus the rock flour that developed during shearing contributed to the low cohesion that was obtained.

69. Three precut specimens of concrete on indurated clay were tested to determine lower bound shear strength parameters. The direct shear test results are presented in Table 4 and in Figure 5. The test results indicate a reasonable failure envelope with an angle of sliding

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* See WES letter report, subject: Preliminary Results, Condition Survey of Lock 8, Monongahela River, Dilliner, Pennsylvania, dated 21 January 1983.

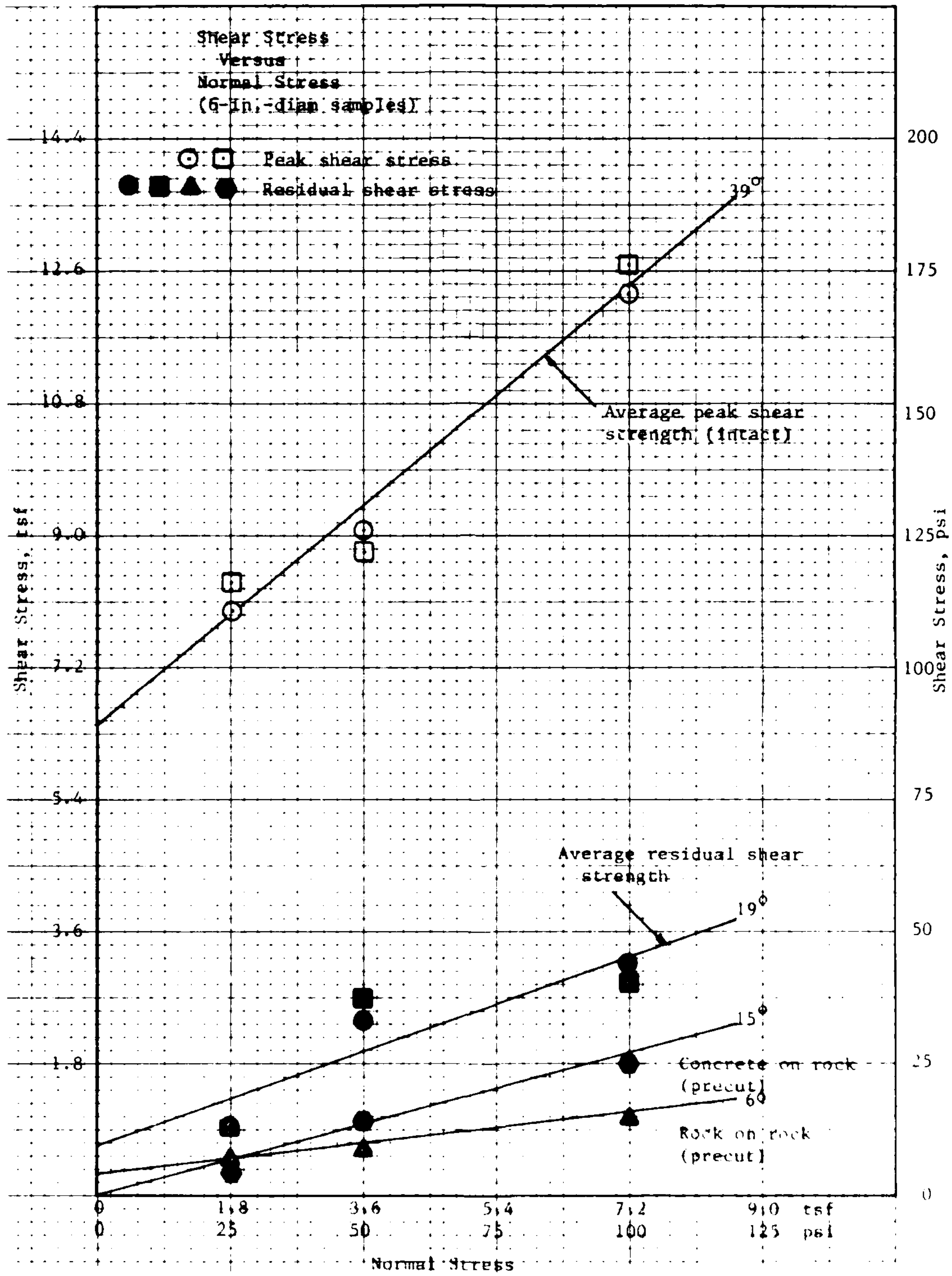

Figure 5. Direct shear test results - failure envelopes, indurated clay, Lock and Dam No. 7, Monongahela River

friction of 15 degrees and a cohesion of zero.

70. Three precut indurated clay specimens were run to determine lower bound shear strength parameters. The test results indicate a relatively low angle of sliding friction of 6 degrees. It does not correlate well with the residual shear strengths obtained on the intact specimens and generally a good correlation is found between these two tests.

71. The direct shear test results obtained on the interbedded sandstone and shale show this rock unit to have shear strengths much higher than the indurated clay. It would also have much higher shear strengths than the shale unit directly above. The ϕ angle of intact specimens is 62 degrees and the cohesion is 17.7 tsf; the ϕ_r is 55 degrees and the cohesion is 1.7 tsf. The ϕ_r of 55 degrees is not typical for sandstone and a value of about 35 degrees is more likely.

Concrete Quality

72. The quality of the concrete in the lock was evaluated using information obtained during the inspection of the lock, field and detailed core logs, petrographic examination results, and characterization and engineering design test results. The quality of the near-surface and internal concrete will be discussed by structural elements, i.e., guide walls, guard walls, land lock wall, and river lock wall. Table 5 presents the characterization and engineering design test results obtained on the concrete core. The stress versus strain graphs obtained on selected concrete cores are presented in Appendix E.

Guide Walls

73. The top surface of the upper and lower guide walls appears to be fair to good quality concrete.

74. Two vertical borings, L-4-82 and L-5-82, were drilled in the lower and upper guide walls, respectively. Beneath the overlay and to a depth of 1.1 ft in L-5-82, the concrete is fractured due to freezing and thawing action. This small zone of damage probably existed prior to

the placement of the overlay. The core from the lower guide wall (L-4-82) reveals subparallel cracking to a depth of 1.6 ft. The cracks are spaced about 0.1 ft apart for the full 1.6 ft. The damage was caused by freezing and thawing action (see Figure 2b). The core beneath the damaged zones appears to be good quality concrete. Presented in Plate 37 is a summary log of borings for cores L-4-82 and L-5-82.

75. Three horizontal borings were drilled in the guide walls, one in the upper and two in the lower guide wall. None of these borings showed signs of deterioration other than on the exposed surface where scaling was evident. The vertical surfaces of the guide walls, as revealed in the three borings, appear to be in good condition. Local areas can be expected to contain some damage due to freezing and thawing; such damage is evident along the walls.

76. Physical property data obtained from pieces of the horizontal cores recovered from the guide walls are presented in Table 5. The lowest and highest compressive strengths for cores from 0.0- to 2.9-ft depth are 3730 and 9150 psi, respectively.

77. The tabulation below summarizes the depth of damaged concrete for the five borings in the guide walls.

Structural Element	Boring No.	Boring Direction Horiz	Boring Direction Vert	Damaged Concrete Depth, ft
Lower guide wall	NG WES L-4-82		X	0.9-1.1
Upper guide wall	NG WES L-5-82		X	0.0-1.6
Upper guide wall	NG PDD Gi-1-82	X		None
Lower guide wall	NG PDD Gi-2-82	X		None
Lower guide wall	NG PDD Gi-3-82	X		None

Guard Walls

78. The top surface of the guard walls appears to be fair to good quality concrete.

79. Two short vertical borings, R-2-82 and R-5-82, were drilled in the upper and lower guard walls, respectively. Core from boring

R-2-82 reveals 0.1 ft of damaged concrete while core from R-5-82 reveals 2.0 ft of concrete in very poor condition. The concrete in the 2-ft interval of core R-5-82 is highly cracked. Many broken surfaces have deposits of alkali-silica gel partially or fully coating them, showing that the concrete was damaged in place. The concrete beneath the damaged zones is good quality material with plus 9000-psi compressive strength. Presented in Plate 38 are summary logs of borings for the R-2-82 and R-5-82 core; Plate 39 shows photographs of portions of these cores.

80. Two horizontal borings, Ga-1-82 and L-17-82, were drilled in the upper and lower guard walls, respectively. Concrete deterioration was observed in the near-surface portion of the core from L-17-82 (0.2 ft); the exposed surface was lightly scaled. Plate 40 is a summary log of borings with selected physical properties of the concrete core from these two borings.

81. The tabulation below summarizes the depth of damaged concrete for the four borings in the guard walls.

Structural Element	Boring No.	Boring Direction Horiz	Boring Direction Vert	Damaged Concrete Depth, ft
Upper guard wall	NG PDD R-2-82		X	0.0-0.1
Lower guard wall	NG PDD R-5-82		X	0.0-2.0
Upper guard wall	NG PDD Ga-1-82	X		None
Lower guard wall	NG PDD L-17-82	X		0.0-0.2

Land Lock Wall

82. The top surface of the lock wall appears to be in poor condition. The three vertical borings in this wall (L-1-82, L-2-82, and L-3-82) show very poor quality concrete (broken and cracked) to a maximum depth of 1.5 ft. Most broken and cracked surfaces have partial or complete coatings of alkali-silica gel. Concrete beneath the damaged zone is considered good quality concrete.

83. Vertical boring L-1-82 is located in the upper miter gate monolith block, L-2-82 at about mid-length of the wall, and L-3-82 about

50 ft upstream of the operations building. Concrete overlay patches 0.1 ft thick were recovered in two borings; local overlay depths can be expected to vary by several tenths of a foot.

84. Boring L-1-82 reveals cracked and broken core to a depth of 1.5 ft. Boring L-2-82 has highly cracked core from 0.1 to 1.4 ft, and the core from L-3-82 contains rubble and highly cracked concrete to 1.4 ft. The concrete physical property test results indicate good quality concrete below these damaged zones. Minimum and maximum strengths are 6,850 and 11,300 psi, respectively; the lower strength concrete was obtained from near the bottom of a boring (38.5 ft). Lock wall thickness is about 42 ft. Plates 41 and 42 illustrate the summary log of borings and selected concrete properties of core from the three vertical borings in the land lock wall. Plate 43 presents photographs of the near-surface core from the three vertical borings in the land lock wall.

85. Five horizontal borings were drilled in the land lock wall, chamber side; three high near upper pool elevation, termed high borings (L-6-82, L-8-82, and L-10-82) and two low near lower pool elevation, termed low borings (L-7-82 and L-9-82). The core from the five borings reveals slight to no near-surface damage; the core from L-7-82 has 0.2 ft of cracking due to freezing and thawing action. Evidence of scaling and erosion is present on the exposed core end. The concrete beyond the exposed surface is of high quality. The five cores beginning at zero depth to 3 ft have a compressive strength range from 7,850 to 10,250 psi; unit weights and compressional wave velocities indicate sound dense concrete for these five cores. The absence of concrete damage in four of these five cores does not mean that near-surface damage is absent in the vertical surface of the land lock wall; Appendix A photographs show local surface areas where damaged concrete exists. It is expected that minor near-surface damage of up to several inches exists locally along the land wall. Plates 44, 45, and 46 illustrate summary logs of borings for the five horizontal borings located in the land lock wall.

86. The tabulation below summarizes the depth of damaged concrete for the eight borings in the land lock wall:

Structural Element	Boring No.	Boring Direction Horiz	Vert	Damaged Concrete Depth, ft
Land lock wall	NG WES L-1-82		X	0.1-1.5
Land lock wall	NG WES L-2-82		X	0.1-1.4
Land lock wall	NG WES L-3-82		X	0.0-1.4
Land lock wall	NG PDD L-6-82	X		None
Land lock wall	NG PDD L-7-82	X		0.0-0.2
Land lock wall	NG PDD L-8-82	X		None
Land lock wall	NG PDD L-9-82	X		None
Land lock wall	NG PDD L-10-82	X		None

River Lock Wall

87. The top surface of the river lock wall appears to be in poor condition. The three vertical cores (R-1-82, R-3-82, and R-4-82) recovered from the wall reveal very poor quality concrete (broken and cracked) to a minimum of 0.9-ft and a maximum of 1.8-ft depth. About 30 percent of the broken surface has partial coatings of alkali-silica gel. Concrete beneath the damage concrete is considered good quality concrete.

88. Vertical boring R-1-82 is in the upper miter gate monolith, R-3-82 is at about mid-length of the lock wall, and R-4-82 is about 85 ft upstream of the lower gate recess. One boring contains a thick overlay. The three cores have 1-1/2- to 2.0-in. maximum size aggregates. The internal concrete has a minimum and maximum strength of 5,130 and 10,220 psi, respectively. These strength values are considered to be representative of the full depth of concrete in the wall; i.e., top of wall to the base of the wall.

89. Plates 47 and 48 present the summary log of borings and selected concrete physical properties for the three vertical borings. Plate 49 presents photographs of the near-surface concrete core from the three vertical borings in the river lock wall.

90. Five horizontal borings were drilled in the river lock wall, chamber side, three high and two low borings; and one was drilled on the river side of the wall. Two of the three cores from the high borings showed concrete damage, while core from the two low borings had no

concrete damage. Maximum depth of damage was 0.9 ft for the high boring located in the lower miter gate recess. The damage observed in the cores is due to subparallel cracking caused by freezing and thawing action. Most of the broken pieces are partially coated with alkali-silica gel indicating that breakage of the concrete occurred in place.

91. Core from boring L-11-82 located in the river side of the wall was not damaged. The author originally located this boring in a severely deteriorated zone and on a wide crack. The L-11-82 boring was not drilled in the area chosen to be representative of the damaged concrete on the river side of the river lock wall. Local concrete damage can be expected to exist on this side of the wall in the severely damaged zones.

92. The internal concrete in the river lock wall is considered to be of good quality. Compressive strengths from pieces of core from the horizontal borings range from 5930 to 9600 psi.

93. Plates 50, 51, and 52 illustrate the summary logs of borings and selected physical properties of core from the six horizontal borings in the river lock wall.

94. The tabulation below summarizes the depth of damaged concrete for the nine borings in the river lock wall:

Structural Element	Boring No. PM-WES	Boring Direction		Damaged Concrete Depth, ft
		Horiz	Vert	
River lock wall	NG WES R-1-82		X	0.0-1.8
River lock wall	NG WES R-3-82		X	0.0-1.3
River lock wall	NG WES R-4-82		X	0.0-0.9
River lock wall	NG PDD L-11-82	X		None
River lock wall	NG PDD L-12-82	X		None
River lock wall	NG PDD L-13-82	X		None
River lock wall	NG PDD L-14-82	X		0.0-0.3
River lock wall	NG PDD L-15-82	X		None
River lock wall	NG PDD L-16-82	X		0.0-0.9

PART VII: CONCLUSIONS, SUMMARY AND RECOMMENDATIONS

Conclusions and Summary

95. The processes causing the greatest amount of distressed and failed concrete are freezing and thawing action and alkali-silica reaction. Both of these processes will continue at accelerated rates if adequate repairs or replacement of the damaged concrete are not carried out.

96. Concrete damage is considered extensive on and near the top surfaces of the guide, guard, and lock walls; it is moderate on vertical surfaces. The top concrete surfaces of the guide, guard, and lock walls are in poor to good condition. Concrete deficiencies are present on all of these top surfaces, e.g., a few small and large spalls, a few areas of light to medium scaling, transverse and longitudinal cracks, and small areas of random and pattern cracking. These defects exist in the concrete overlays and in the original concrete. Extensive patching is present on the top surface of the lock walls with many being drummy.

97. Low quality concrete to a minimum depth of 0.1 ft and to a maximum depth of 2.0 ft was observed in 90 percent of the vertical borings placed in the guide, guard, and lock walls. As observed in the vertical borings that contained damaged concrete, the average depth of low quality concrete is 1.3 ft. The low quality concrete is present as either rubble, broken pieces less than 3 in. in size, or cracked pieces 3 to 10 in. thick.

98. The vertical surfaces of the guide, guard, and lock walls above upper and lower pool elevations are in fair to poor condition. The land and river chamber walls are severely scaled; no uniform depth of scaling was measured. Local large spalls are present. Most of the vertical monolith joints are spalled from lower pool elevation to the corner armor; 6- to 12-in.-deep spalls are common in these joints.

99. Low quality concrete to a minimum depth of 0.2 ft and to a maximum depth of 0.9 ft was observed in 25 percent of the horizontal borings placed in the guide, guard, and lock walls. As observed in the horizontal borings that contained damaged concrete, the average depth of

of low quality concrete is 0.4 ft. The physical condition of the low quality concrete in the horizontal borings is the same as observed in the vertical borings. The vertical surfaces at the lock are not damaged to the extent that the vertical surfaces at Lock 8 are.

100. For those low quality concrete zones where rubble and pieces <3 in. in size occur, this material occupies about 40 percent of the zone. The remainder of the zone contains cracked concrete resulting in pieces of core ranging from 3 to 10 in. thick. The majority of the cores revealed subparallel cracking due to freezing and thawing action.

101. The lower miter-gate recess in the river wall contains broken and cracked concrete. The cracked and broken near-surface concrete is subject to further deterioration by continued freezing and thawing and possible alkali-silica reaction.

102. About one-half of the broken and cracked surfaces of concrete core were partially or fully coated with alkali-silica gel, thus showing damage occurred in place. This white reaction product was observed throughout all cores; however, it decreased with depth in the vertical borings. It was not within the scope of this investigation to determine if the alkali-silica reactivity is continuing or has terminated. If it is ongoing, then it is assumed to be occurring slowly.

103. Infiltrating water into the scaled and spalled areas and cracked concrete at the lock will continue to cause critical saturation levels in the concrete. As this level is reached, and freezing and thawing action occurs, the concrete will be further deteriorated.

104. The ability of the surface and near-surface concrete to perform satisfactorily under anticipated conditions of future service is in question, especially on horizontal surfaces. It is now not performing its originally intended purpose.

105. Concrete beneath the damaged zones is considered to be of good quality. The lowest compressive strength obtained on concrete core from internal sections of the guide, guard, and lock walls is 3730 psi. The average compressive strength of 30 cores is 8320 psi with a standard deviation of 1737 psi. Indications are that the internal concrete should remain in serviceable condition for a period extending on the order of

50 years. The average strength of 30 concrete cores from the Lock 8 structure was 4870 psi. The higher-strength concrete in the Lock 7 structure may be one reason why the extent of concrete damage is not as extensive at Lock 7 as it is at the Lock 8 structure.

106. The lithologic units identified in the three deep borings consist of flat lying cyclic sediments. The rocks encountered were shale, interbedded shale, siltstone and sandstone, sandstone, and an indurated clay. No significant lithologic differences exist nor are any structural feature differences apparent between the three borings. The three vertical borings through the lock walls show the walls to be founded on the moderately hard shale. A high percentage of this rock unit was broken; probably it was broken in place and during the drilling operation.

107. Direct-shear test results were obtained on indurated clay samples recovered from a bed some 18 ft below the base of the lock walls. The rock core recovered from the shale units directly beneath the lock walls was fractured such that intact test specimens could not be obtained. The petrographic examination indicated a marked similarity between the indurated clay and the shale. It is reasonable to assume that the shear strength differences between these two rocks will not be great because of these similarities. Until the shale is resampled and tested, the shear strengths obtained from the indurated clay can be used for preliminary calculations of sliding stability analysis. The average peak shear strengths are $\phi = 39$ degrees and $c = 6.4$ tsf and the average residual shear strengths are $\phi_r = 19$ degrees and $c = 0$.

Recommendations

108. It is recommended that the shale directly beneath the lock walls be resampled and tested in direct shear.

109. If the lock is to be rehabilitated, it is recommended that all damaged concrete be removed to its full depth and replaced by high quality frost-resistant concrete.

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Table 1

Engineering Information and Data,

Lock No. 7, Mon River

File #M-70-4	Plan, Elevation & Sections
M-70-5	Plan, Elevation & Sections
M-54-14	Borings
M-72-1	Lock Walls, Plans, Elevations & Sections
M-72-2	Guide, Guard & Core Walls
M-74-4	Upper Guide Wall Extension
M-74-1	Valves
MSC-596	Repairs-Upper Guard Wall

Charts 13
Precise Alignment & Settlement - Land Wall & River Wall
Piezometer Readings 1979 thru 1982
Periodic Inspection Reports L/D 7, Reports 1, 2, and 3
Color Photograph L/D 7

Table 2

Pertinent Boring Information

Boring No. NG-WES	Monolith No.	Boring Depth, ft	Core Size, in.	Direction of Boring	Elev Top of Boring, ft	Elev Top of Rock, ft	Elev Bottom of Boring, ft
L-1-82	L-19	72.8	6	V	788.0	744.9	715.6
L-2-82	L-24	6.2	4	V	788.0		781.8
L-3-82	L-27	4.7	4	V	788.0		783.3
L-4-82	L-36	2.9	6	V	788.0		785.1
L-5-82	L-6	3.0	6	V	788.0		785.0
R-1-82	R-4	72.9	6	V	788.0	746.4	714.9
R-2-82	R-2	8.4	6	V	788.0		779.6
R-3-82	R-9	75.0	4	V	788.0	747.3	713.0
R-4-82	R-12	8.1	4	V	788.0		779.9
R-5-82	R-16	9.8	4	V	788.0		778.2
NG-PDD							
L-6-82	L-13	1.5	6	H	781.0		
L-7-82	L-13	2.7	6	H	768.7		
L-8-82	L-17	3.1	6	H	783.0		
L-9-82	L-18	3.1	6	H	768.7		
L-10-82	L-22	3.1	6	H	783.0		
L-11-82	R-4	3.2	6	H	781.0		
L-12-82	R-4	3.1	6	H	781.0		
L-13-82	R-4	2.9	6	H	768.7		
L-14-82	R-11	3.1	6	H	785.0		
L-15-82	R-13	3.2	6	H	768.7		
L-16-82	R-14	3.0	6	H	783.0		
L-17-82	R-14	3.0	6	H	776.0		
Ga-1-82	R-2	3.0	6	H	783.5		
Gi-1-82	L-1	2.7	6	H	786.0		
Gi-2-82	L-25	2.9	6	H	768.7		
Gi-3-83	L-30	3.2	6	H	768.7		

Table 3

Pertinent Core Information

Boring No.	Box No.	Depth, ft	Elevation Top of Boring	Elevation Top of Rock	Elevation Bottom of Boring	Structures Laboratory Identification No.
NG-WES L-1-82	1 of 19	0.0- 3.5	788.0	744.9	715.6	PITT-10 CON-62
	2 of 19	3.5- 8.1				PITT-10 CON-63
	3 of 19	8.1-12.7				PITT-10 CON-64
	4 of 19	12.7-16.8				PITT-10 CON-65
	5 of 19	16.8-21.4				PITT-10 CON-66
	6 of 19	21.4-25.8				PITT-10 CON-67
	7 of 19	25.8-30.2				PITT-10 CON-68
	8 of 19	30.2-34.0				PITT-10 CON-69
	9 of 19	34.0-37.8				PITT-10 CON-70
	10 of 19	37.8-41.7				PITT-10 CON-71
	11 of 19	41.7-46.3				PITT-10 DC-25
	12 of 19	46.3-49.7				PITT-10 DC-26
	13 of 19	49.7-53.9				PITT-10 DC-27
	14 of 19	53.9-56.3				PITT-10 DC-28
	15 of 19	56.3-58.8				PITT-10 DC-29
	16 of 19	58.8-61.5				PITT-10 DC-30
	17 of 19	61.5-65.3				PITT-10 DC-31
	18 of 19	65.3-68.4				PITT-10 DC-32
	19 of 19	68.4-72.8				PITT-10 DC-33
NG-WES L-2-82	1 of 1	0.0- 6.2	788.0		781.8	PITT-10 CON-72
NG-WES L-3-82	1 of 1	0.0- 4.7	788.0		783.3	PITT-10 CON-73
NG-WES L-4-82	1 of 1	0.0- 2.9	788.0		785.1	PITT-10 CON-74
NG-WES L-5-82	1 of 1	0.0- 3.0	788.0		785.0	PITT-10 CON-75
PDD L-6-82	1 of 1	0.0- 1.5	781.0			PITT-10 CON-108
PDD L-7-82	1 of 1	0.0- 2.7	786.7			PITT-10 CON-107

(Continued)

Table 3 (Continued)

Boring No.	Box No.	Depth, ft	Elevation Top of Boring	Elevation Top of Rock	Elevation Bottom of Boring	Structures Laboratory Identification No.
PDD L-8-82	1 of 1	0.0- 3.1	783.0			PITT-10 CON-117
PDD L-9-82	1 of 1	0.0- 3.1	768.7			PITT-10 CON-114
PDD L-10-82	1 of 1	0.0- 3.1	783.0			PITT-10 CON-104
PDD L-11-82	1 of 1	0.0- 3.2	781.0			PITT-10 CON-101
PDD L-12-82	1 of 1	0.0- 3.1	781.0			PITT-10 CON-111
PDD L-13-82	1 of 1	0.0- 2.9	768.7			PITT-10 CON-110
PDD L-14-82	1 of 1					PITT-10 CON-120
PDD L-15-82	1 of 1	0.0- 3.2	768.7			PITT-10 CON-106
PDD L-16-82	1 of 1	0.0- 3.0	785.0			PITT-10 CON-116
PDD L-17-82	1 of 1	0.0- 3.0	783.0			PITT-10 CON-112
PDD L-18-82	1 of 1	0.0- 3.1	776.0			PITT-10 CON-103
NG-WES R-1-82	1 of 20	0.0- 3.8	788.0	746.4	714.9	PITT-10 CON-76
	2 of 20	3.8- 8.6				PITT-10 CON-77
	3 of 20	8.6-11.1				PITT-10 CON-78
	4 of 20	11.1-14.4				PITT-10 CON-79
	5 of 20	14.4-18.4				PITT-10 CON-80
	6 of 20	18.4-22.5				PITT-10 CON-81
	7 of 20	22.5-24.8				PITT-10 CON-82
	8 of 20	24.8-28.6				PITT-10 CON-83
	9 of 20	28.6-33.3				PITT-10 CON-84
	10 of 20	33.3-37.3				PITT-10 CON-85
	11 of 20	37.3-41.7				PITT-10 CON-86
	12 of 20	41.7-44.7				PITT-10 DC-34
	13 of 20	44.7-48.2				PITT-10 DC-35
	14 of 20	48.2-51.1				PITT-10 DC-36

(Continued)

Table 3 (Continued)

Boring No.	Box No.	Depth, ft	Elevation Top of Boring	Elevation Top of Rock	Elevation Bottom of Boring	Structures Laboratory Identification No.
NG-WES R-1-82	15 of 20	51.1-54.9				PITT-10 DC-37
	16 of 20	54.9-58.9				PITT-10 DC-38
	17 of 20	58.9-62.9				PITT-10 DC-39
	18 of 20	62.9-67.2				PITT-10 DC-40
	19 of 20	67.2-69.3				PITT-10 DC-41
	20 of 20	69.3-72.9				PITT-10 DC-42
NG-WES R-2-82	1 of 2	0.0- 4.1	788.0		779.6	PITT-10 CON-87
	2 of 2	4.1- 8.4				PITT-10 CON-88
NG-WES R-3-82	1 of 14	0.0- 5.4	788.0	747.3	713.0	PITT-10 CON-89
	2 of 14	5.4- 7.6				PITT-10 CON-90
	3 of 14	7.6-11.5				PITT-10 CON-91
	4 of 14	11.5-16.4				PITT-10 CON-92
	5 of 14	16.4-21.0				PITT-10 CON-93
	6 of 14	21.0-26.1				PITT-10 CON-94
	7 of 14	26.1-33.5				PITT-10 CON-95
	8 of 14	33.5-40.7				PITT-10 CON-96
	9 of 14	40.7-46.0				PITT-10 DC-43
	10 of 14	46.0-50.6				PITT-10 DC-44
	11 of 14	50.6-57.9				PITT-10 DC-45
	12 of 14	57.9-63.6				PITT-10 DC-46
	13 of 14	63.6-70.1				PITT-10 DC-47
	14 of 14	70.1-75.0				PITT-10 DC-48
NG-WES R-4-82	1 of 2	0.0- 6.1	788.0		779.9	PITT-10 CON-97
	2 of 2	6.1- 8.1				PITT-10 CON-98
NG-WES R-5-82	1 of 2	0.0- 8.7	788.0		778.2	PITT-10 CON-99
	2 of 2	8.7- 9.8				PITT-10 CON-100
PDD Ga-1-82	1 of 1	0.0- 3.0	783.5			PITT-10 CON-105

(Continued)

Table 3 (Concluded)

Boring No.	Box No.	Depth, ft	Elevation Top of Boring	Elevation Top of Rock	Elevation Bottom of Boring	Structures Laboratory Identification No.
PDD Gi-1-82	1 of 1	0.0- 2.7	786.0			PITT-10 CON-113
PDD Gi-2-82	1 of 1	0.0- 2.9	768.7			PITT-10 CON-109
PDD Gi-3-82	1 of 1	0.0- 3.0	768.7			PITT-10 CON-115

Table 4

Lock and Dam No. 7

Summary Direct-Shear Test Results

Rock Type and Test Type	Boring No. NG WES	Depth, ft	Normal Stress, tsf	Peak Shear Stress, tsf	Peak Shear Strength	Residual Shear Stress, tsf	Residual Shear Strength
Indurated clay, soft to mod. hard, gray INTACT	R-1-82	70.7-71.1	1.8	8.0	$\phi = 39^o$	1.0	$\phi_r = 21^o$
	R-1-82	67.5-67.8	3.6	9.1	$c = 6.4$ tsf	2.4	$c = 0.6$ tsf
	L-1-82	66.7-66.9	7.2	12.3		3.2	
					Avg $\phi = 39^o$		Avg $\phi_r = 19^o$
					$c = 6.4$ tsf		$c = 0.7$ tsf
Indurated clay, soft to mod. hard, gray INTACT	R-1-82	70.4-70.7	1.8	8.4	$\phi = 40^o$	0.9	$\phi_r = 18^o$
	R-1-82	70.1-70.4	3.6	8.8	$c = 6.5$ tsf	2.7	$c = 0.8$ tsf
	L-1-82	72.0-72.3	7.2	12.7		2.9	
Indurated clay, soft to mod. hard, gray PRECUT RK on RK	R-1-82	69.8-70.1	1.8			0.5	$\phi_r = 6^o$
	R-1-82	69.5-69.8	3.6			0.6	$c = 0.3$ tsf
	R-1-82	67.8-68.1	7.2			1.0	
Indurated clay, soft to mod. hard, gray PRECUT CON on RK	L-1-82	68.0-68.2	1.8			0.3	$\phi_r = 15^o$
	L-1-82	67.0-67.1	3.6			1.0	$c = 0$
	L-1-82	67.1-67.3	7.2			1.8	
Interbedded sandstone and shale, hard, lt gray INTACT	R-1-82	53.0-53.4	1.8	16.6	$\phi = 62^o$	1.6	$\phi_r = 55^o$
	R-1-82	53.4-53.8	3.6	31.2	$c = 17.7$ tsf	10.8	$c = 1.7$ tsf
	R-1-82	53.9-54.3	7.2	29.0		10.6	

Table 5

Concrete Core Test Results, Lock No. 7, Monongahela River

Boring No.	Depth of Core, ft	Characterization Properties: Unit Weight, pcf	Characterization Properties: Comp Wave Velocity, fps	Characterization Properties: Comp Strength, psi	Engineering Design Properties: Elastic Modulus psi X 10^6	Engineering Design Properties: Poisson's Ratio
NG WES L-1-82	1.7- 2.7	151.1	14,877	11,300	4.80	0.15
NG WES L-1-82	20.0-21.0	150.0	15,199	8,950	4.69	0.16
NG WES L-1-82	38.0-39.0	147.8	14,296	6,850	4.15	0.18
NG PDD L-6-82	0.1- 1.0	149.2	15,000	10,000	7.87	0.24
NG PDD L-7-82	0.2- 1.1	148.8	13,681	8,050	3.99	0.12
NG PDD L-8-82	0.0- 1.0	148.0	14,285	9,030	4.94	0.16
NG PDD L-8-82	2.0- 3.0	149.2	14,309	7,850		
NG PDD L-9-82	0.0- 1.0	152.4	14,962	10,640		
NG PDD L-10-82	0.1- 1.1	149.4	15,214	10,250	5.48	0.16
NG PDD L-10-82	2.0- 3.0	148.8	13,835	10,150	4.29	0.16
NG PDD L-11-82	0.0- 1.0	151.8	--			
NG PDD L-11-82	2.2- 3.2	150.6	--			
NG PDD L-12-82	0.0- 1.0	149.9	14,900	9,600	4.93	0.17
NG PDD L-12-82	2.1- 3.1	150.5	14,825	7,900	5.04	0.19
NG PDD L-13-82	0.1- 1.1	149.5	--			
NG PDD L-13-82	2.1- 3.1	148.7	14,800	9,440		
NG PDD L-14-82	0.2- 1.2	147.8	11,724			
NG PDD L-14-82	1.9- 2.9	148.5	14,669	8,690		
NG PDD L-15-82	0.1- 1.1	146.8	12,799	6,990		
NG PDD L-15-82	2.1- 3.1	148.0	14,226	7,850		
NG PDD L-16-82	2.0- 3.0	142.8	13,400	5,930	4.90	0.17
NG PDD L-17-82	0.2- 1.2	146.3	13,179	6,150	3.54	0.23
NG WES R-1-82	1.8- 2.8	149.0	14,432	7,400	3.36	0.14
NG WES R-1-82	21.4-22.4	149.6	14,888	9,000	4.62	0.21
NG WES R-1-82	39.0-40.0	145.8	14,061	5,130	2.77	0.10
NG WES R-2-82	1.3- 2.2	154.4	14,705	9,080	4.51	0.15
NG WES R-2-82	6.4- 7.4	149.4	14,202	9,530	4.20	0.16

(Continued)

Table 5 (Concluded)

		Characterization Properties			Engineering Design Properties	
Boring No.	Depth of Core, ft	Unit Weight, pcf	Comp Wave Velocity, fps	Comp Strength, psi	Elastic Modulus psi X 10^6	Poisson's Ratio
NG WES R-3-82	1.8- 2.8	150.1	14,988	7,450	4.79	0.14
NG WES R-3-82	19.5-20.2	150.3	14,493	8,300	4.62	0.15
NG WES R-3-82	37.2-37.9	149.3	14,113	6,200	4.07	0.13
NG PDD Ga-1-82	0.0- 1.0	151.3	14,396	10,220		
NG PDD Gi-1-82	0.0- 1.0	141.2	12,195	3,730	2.68	0.13
NG PDD Gi-2-82	0.0- 1.0	146.8	14,250	9,150	4.98	0.16
NG PDD Gi-2-82	1.9- 2.9	149.7	15,[illegible]3	8,900	5.30	0.16

R/H 21
LOCK & DAM 7
MONONGAHELA RIVER

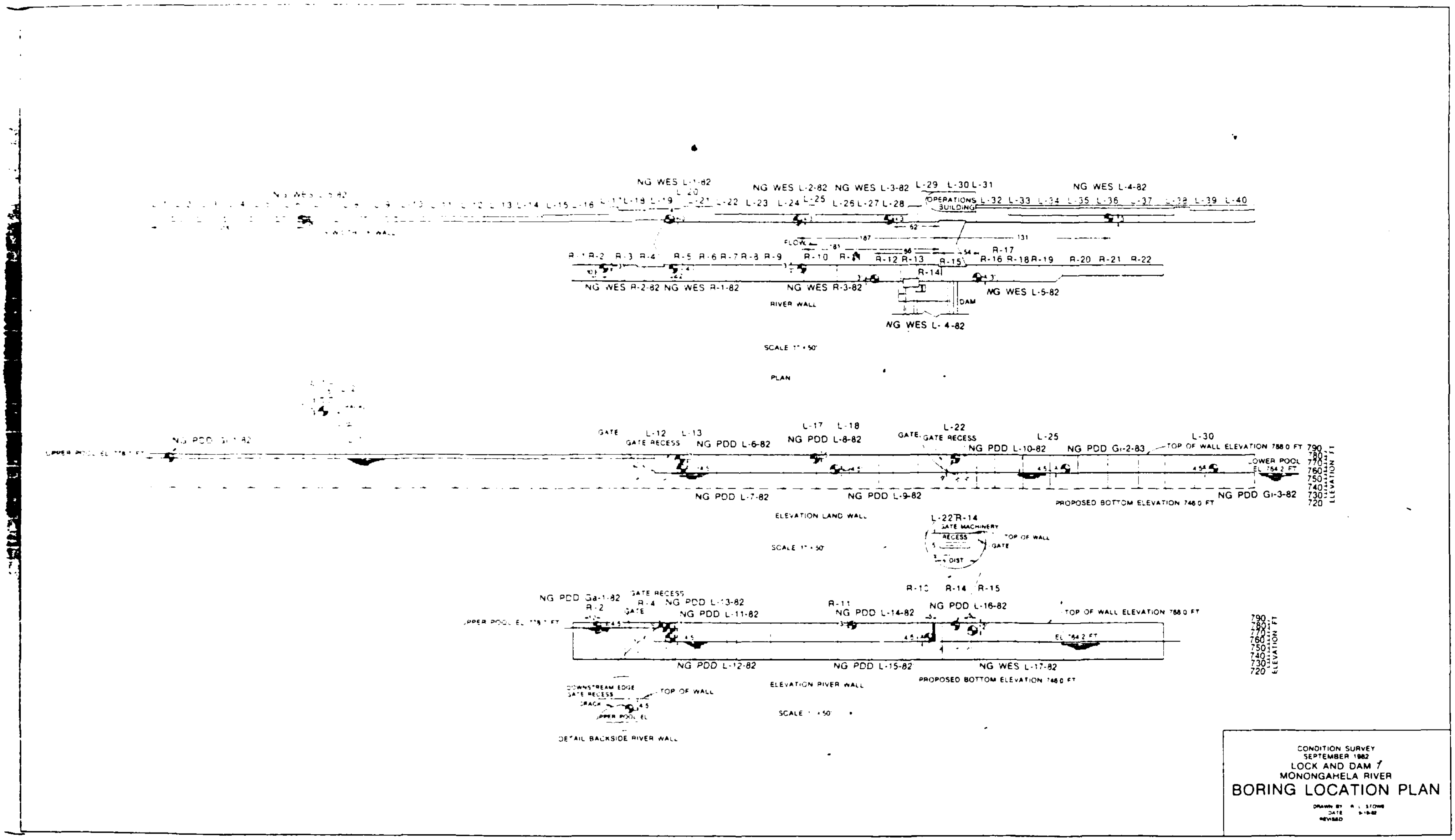
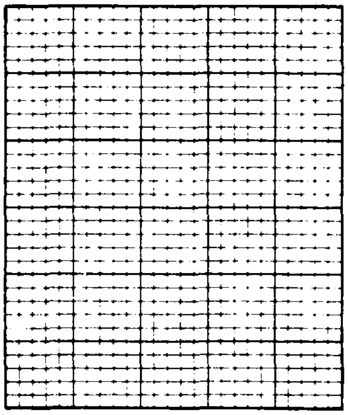
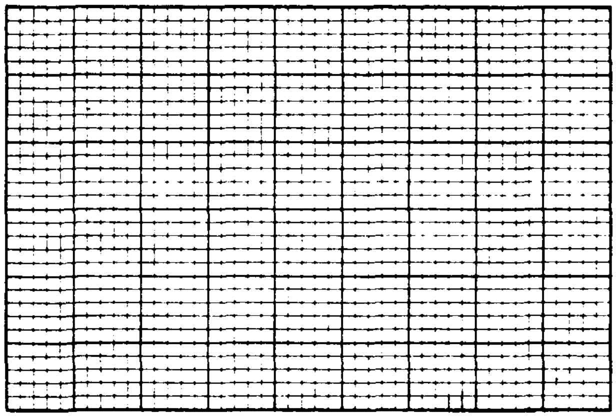
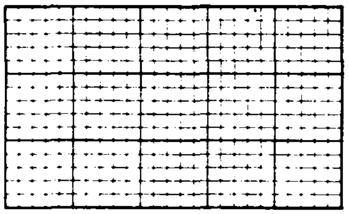

NG WES L-1-82
NG WES L-2-82
NG WES L-3-82
NG WES L-4-82
OPERATIONS BUILDING
FLOW
NG WES R-2-82
NG WES R-1-82
NG WES R-3-82
NG WES L-5-82
NG WES L-4-82
DAM
RIVER WALL
SCALE 1" = 50'
PLAN
GATE
GATE RECESS
NG PDD L-6-82
NG PDD L-8-82
NG PDD L-10-82
NG PDD Gi-2-83
TOP OF WALL ELEVATION 788.0 FT
LOWER POOL EL 754.2 FT
NG PDD L-7-82
NG PDD L-9-82
NG PDD Gi-3-82
PROPOSED BOTTOM ELEVATION 746.0 FT
ELEVATION, FT
ELEVATION LAND WALL
SCALE 1" = 50'
GATE MACHINERY RECESS
TOP OF WALL
NG PDD Ga-1-82
NG PDD L-13-82
NG PDD L-11-82
NG PDD L-14-82
NG PDD L-16-82
NG PDD L-12-82
NG PDD L-15-82
NG WES L-17-82
ELEVATION RIVER WALL
DOWNSTREAM EDGE GATE RECESS
CRACK
DETAIL BACKSIDE RIVER WALL
CONDITION SURVEY
SEPTEMBER 1982
LOCK AND DAM 7
MONONGAHELA RIVER
BORING LOCATION PLAN

Plate 2







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. Boring No. NG WES Depth, ft		R-1-82 70.7	R-1-82 67.5	L-1-82 66.7	R-1-82 70.4	R-1-82 70.1	L-1-82 72.0
WET DENSITY, PCF	γ_d	154.7	164.4	155.7	154.7	155.0	155.2
WATER CONTENT	w	7.9%	6.5%	7.9%	7.1%	6.4%	7.5%
NORMAL STRESS, TSF	σ	1.8	3.6	7.2	1.8	3.6	7.2
MAXIMUM SHEAR STRESS, TSF	τ_f	8.0	9.1	12.3	8.4	8.8	12.7
TIME TO FAILURE, MINUTES	t_f	12.0	21.0	25.0	15.0	20.0	28.0
ULTIMATE SHEAR STRESS, TSF	τ_r	1.0	2.4	3.2	0.9	2.7	2.9
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL Indurated clay, soft to mod. hard, gray

REMARKS

PROJECT Lock and Dam 7, Mon River

INTACT

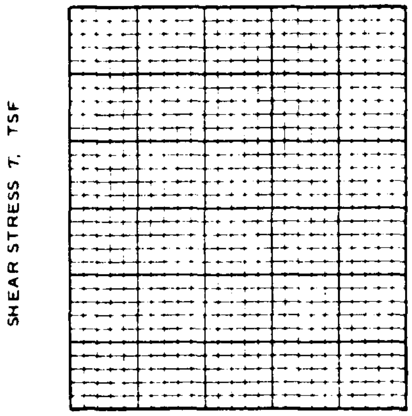
AREA

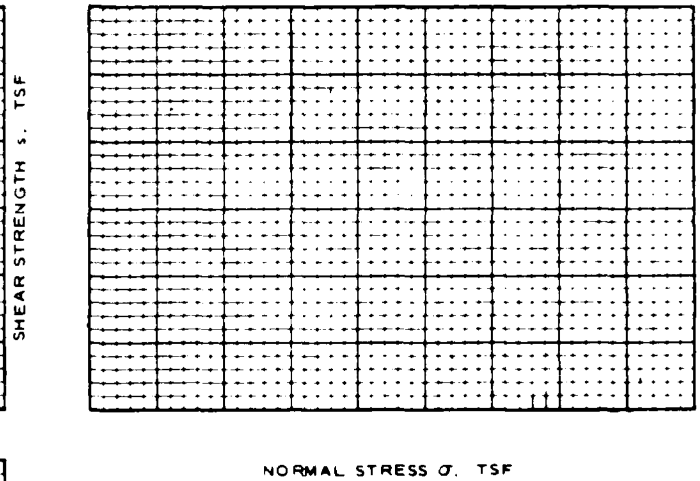
BORING NO. See Test No. SAMPLE NO.

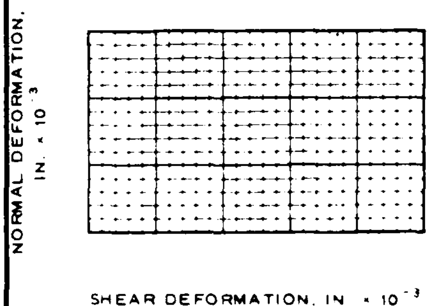
DEPTH EL See Test No. DATE 11-26 Feb 1983

DIRECT SHEAR TEST REPORT (ROCK)

Plate 3







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. Boring No. NG WES Depth, ft		L-1-82 68.0	L-1-82 67.0	L-1-82 67.1			
WET DENSITY, PCF	γ_d	156.6	156.5	165.4			
WATER CONTENT	w	6.5%	6.7%	5.9%	%	%	%
NORMAL STRESS, TSF	σ	1.8	3.6	7.2			
MAXIMUM SHEAR STRESS, TSF	τ_f	0.4	1.3	2.6			
TIME TO FAILURE, MINUTES	t_f	5.0	8.0	14.0			
ULTIMATE SHEAR STRESS, TSF	τ_r	0.3	1.0	1.8			
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL Indurated clay, soft to mod. hard, gray

REMARKS

PROJECT Lock and Dam 7, Mon River

PRECUT, concrete on rock

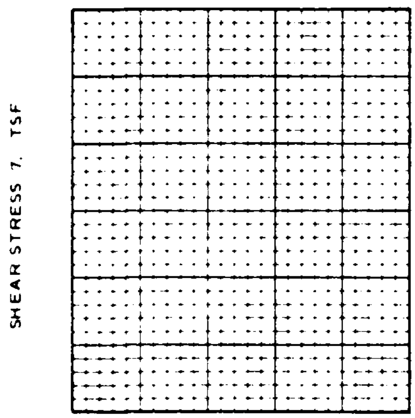
AREA

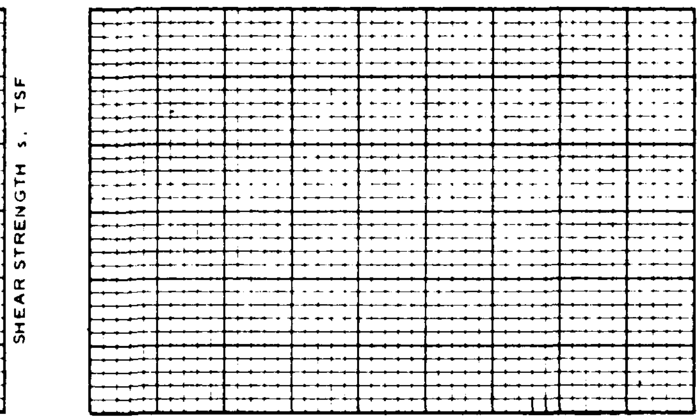
BORING NO. See Test No. SAMPLE NO.

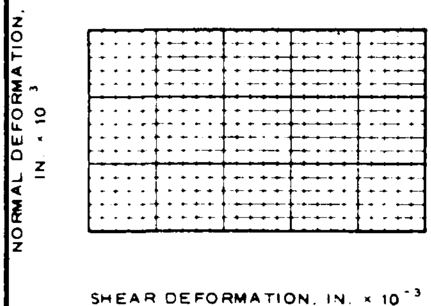
DEPTH EL See Test No. DATE 11-26 Feb 1983

DIRECT SHEAR TEST REPORT (ROCK)

Plate 4







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. Boring No. NG WES Depth, ft		R-1-82 69.8	R-1-82 69.5	R-1-82 67.8			
WET DENSITY, PCF	γ_d	157.3	157.7	158.0			
WATER CONTENT	w	5.8%	5.4%	5.4%	%	%	%
NORMAL STRESS, TSF	σ	1.8	3.6	7.2			
MAXIMUM SHEAR STRESS, TSF	τ_f	0.5	1.0	1.9			
TIME TO FAILURE, MINUTES	t_f	10.0	7.0	9.0			
ULTIMATE SHEAR STRESS, TSF	τ_r	0.5	0.6	1.0			
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL Indurated clay, soft to mod. hard, gray

REMARKS

PROJECT Lock and Dam 7, Mon River

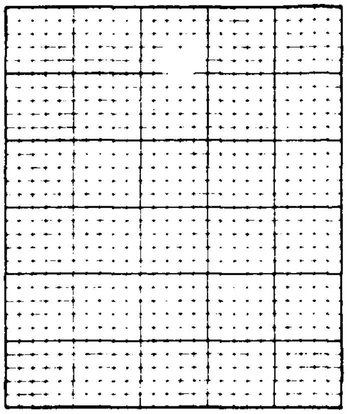
PRECUT, rock on rock

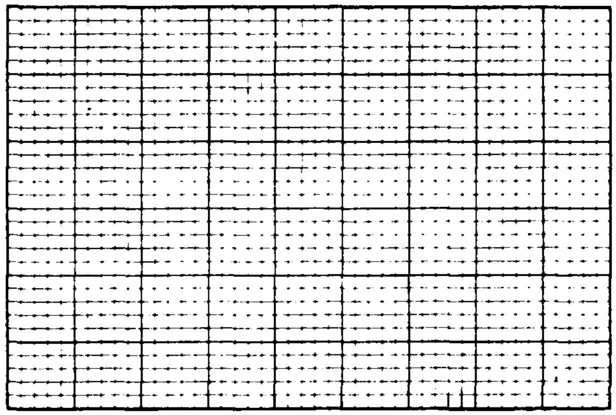
AREA

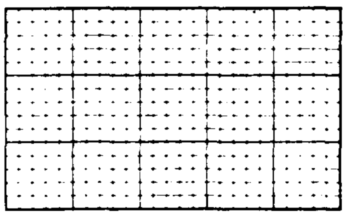
BORING NO. See Test No. SAMPLE NO.

DEPTH EL See Test No. DATE 11-26 Feb 1983

DIRECT SHEAR TEST REPORT (ROCK)







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

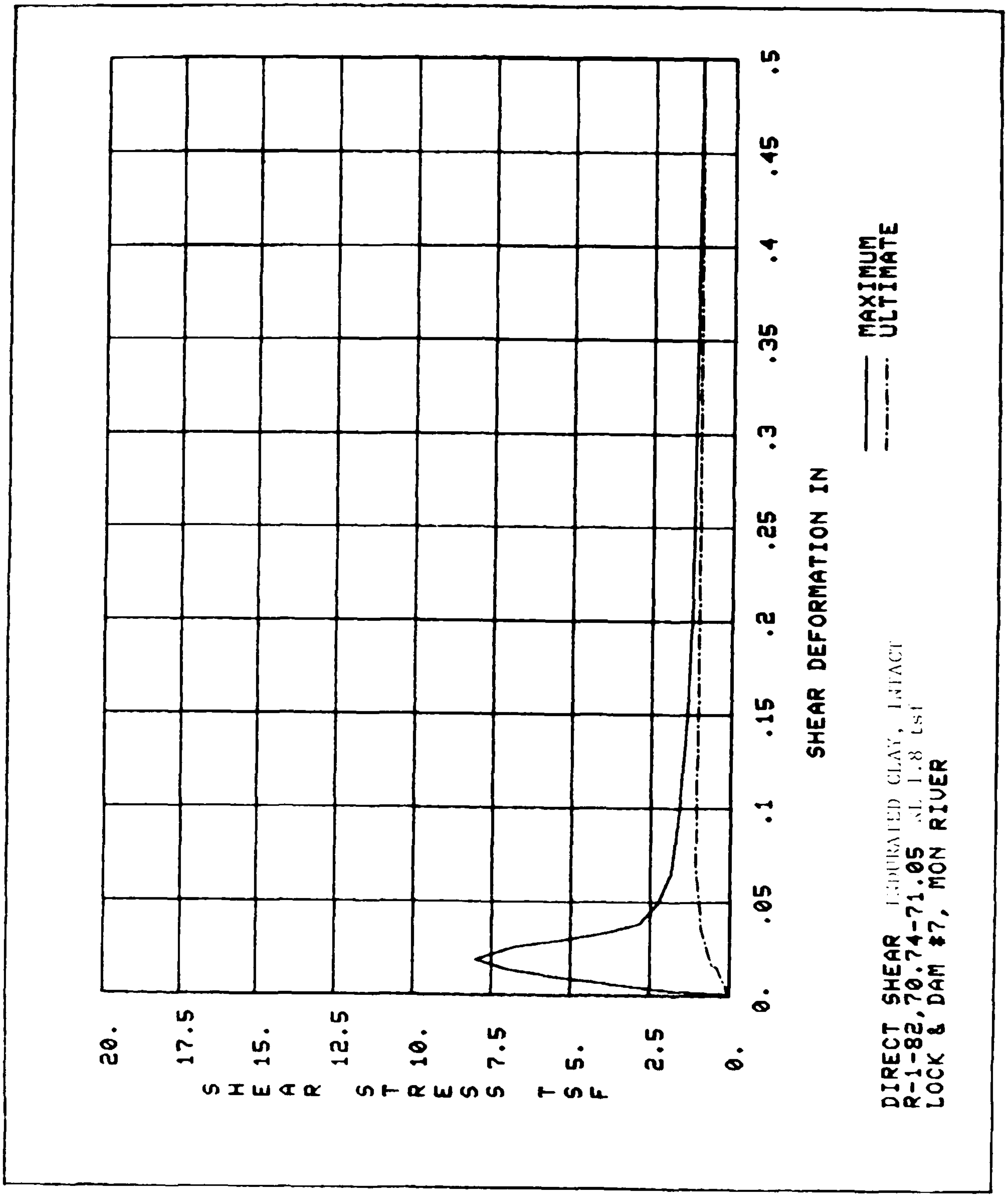
TEST NO. Boring No. NG WES Depth, ft		R-1-82 53.0	R-1-82 53.4	R-1-82 53.9			
WET DENSITY, PCF	γ_d	163.1	164.1	167.5			
WATER CONTENT	w	5.2 %	3.0 %	4.6 %	%	%	%
NORMAL STRESS, TSF	σ	1.8	3.6	7.2			
MAXIMUM SHEAR STRESS, TSF	τ_f	16.6	31.2	29.0			
TIME TO FAILURE, MINUTES	t_f	20.0	45.0	31.0			
ULTIMATE SHEAR STRESS, TSF	τ_r	1.6	10.8	10.6			
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

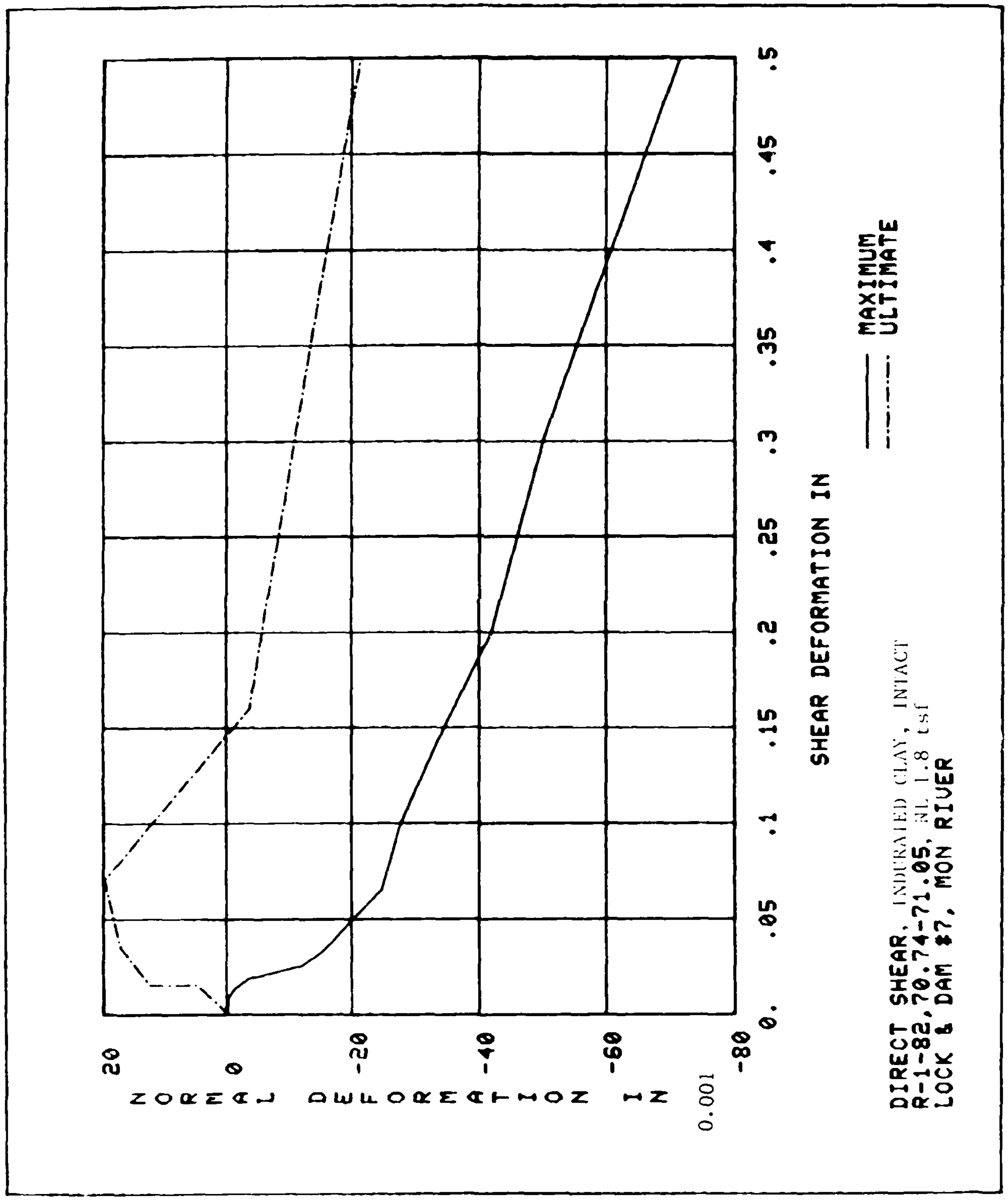
DESCRIPTION OF MATERIAL Interbedded sandstone and shale, hard, lt gray

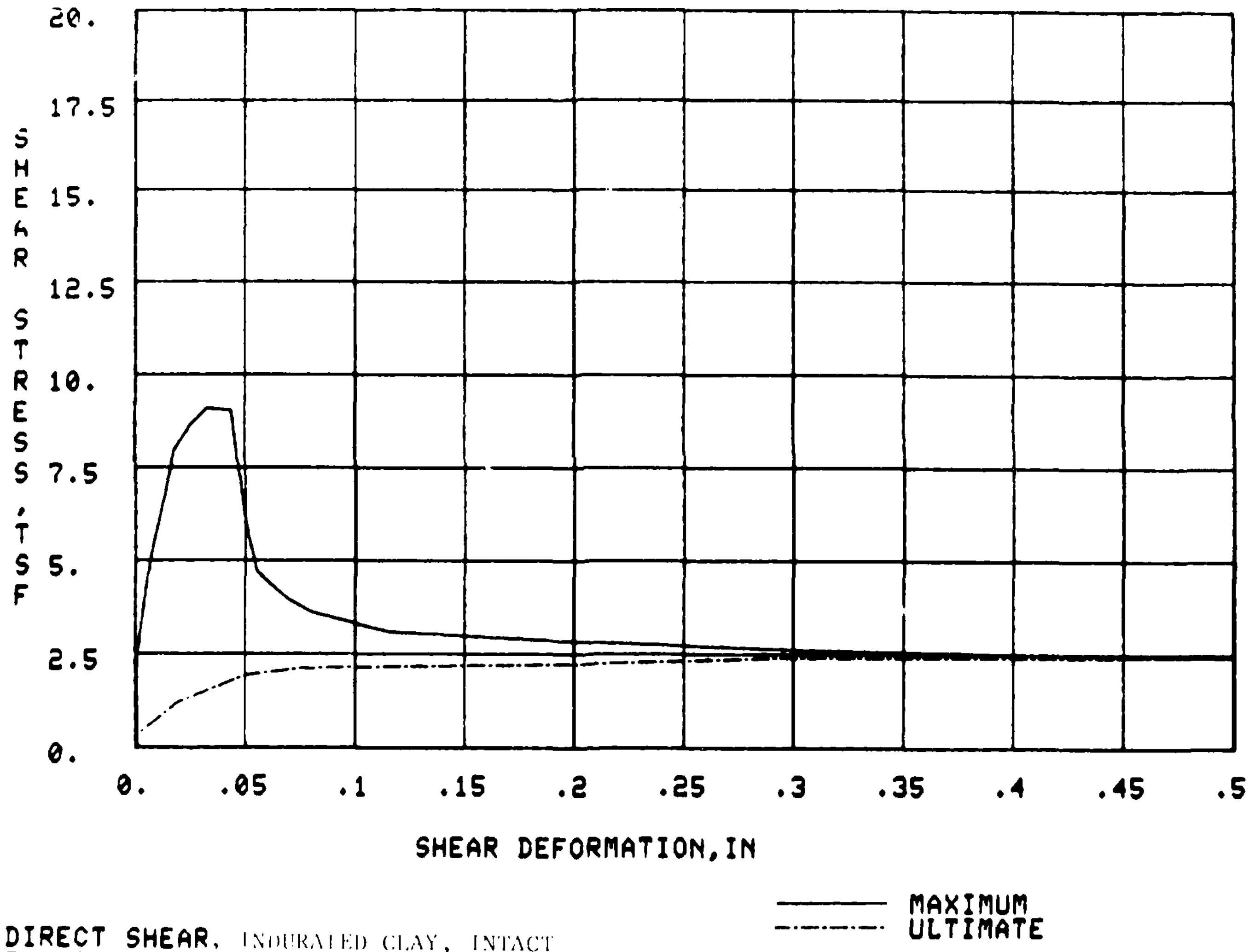
REMARKS

PROJECT Lock and Dam 7, Mon River	
INTACT	
AREA	
BORING NO. See Test No.	SAMPLE NO.
DEPTH EL See Test No.	DATE 11-26 Feb 1983

DIRECT SHEAR TEST REPORT (ROCK)


SHEAR STRESS TSF
20.
17.5
15.
12.5
10.
7.5
5.
2.5
0.
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
ULTIMATE
DIRECT SHEAR INDURATED CLAY, INTACT
R-1-82,70.74-71.05 NL 1.8 tsf
LOCK & DAM #7, MON RIVER


NORMAL DEFORMATION IN 0.001
20
0
-20
-40
-60
-80
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
ULTIMATE
DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82,70.74-71.05, NL 1.8 tsf
LOCK & DAM #7, MON RIVER



DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82,67.54-67.80, NL 3.6 tsf
LOCK & DAM 7, MON RIVER

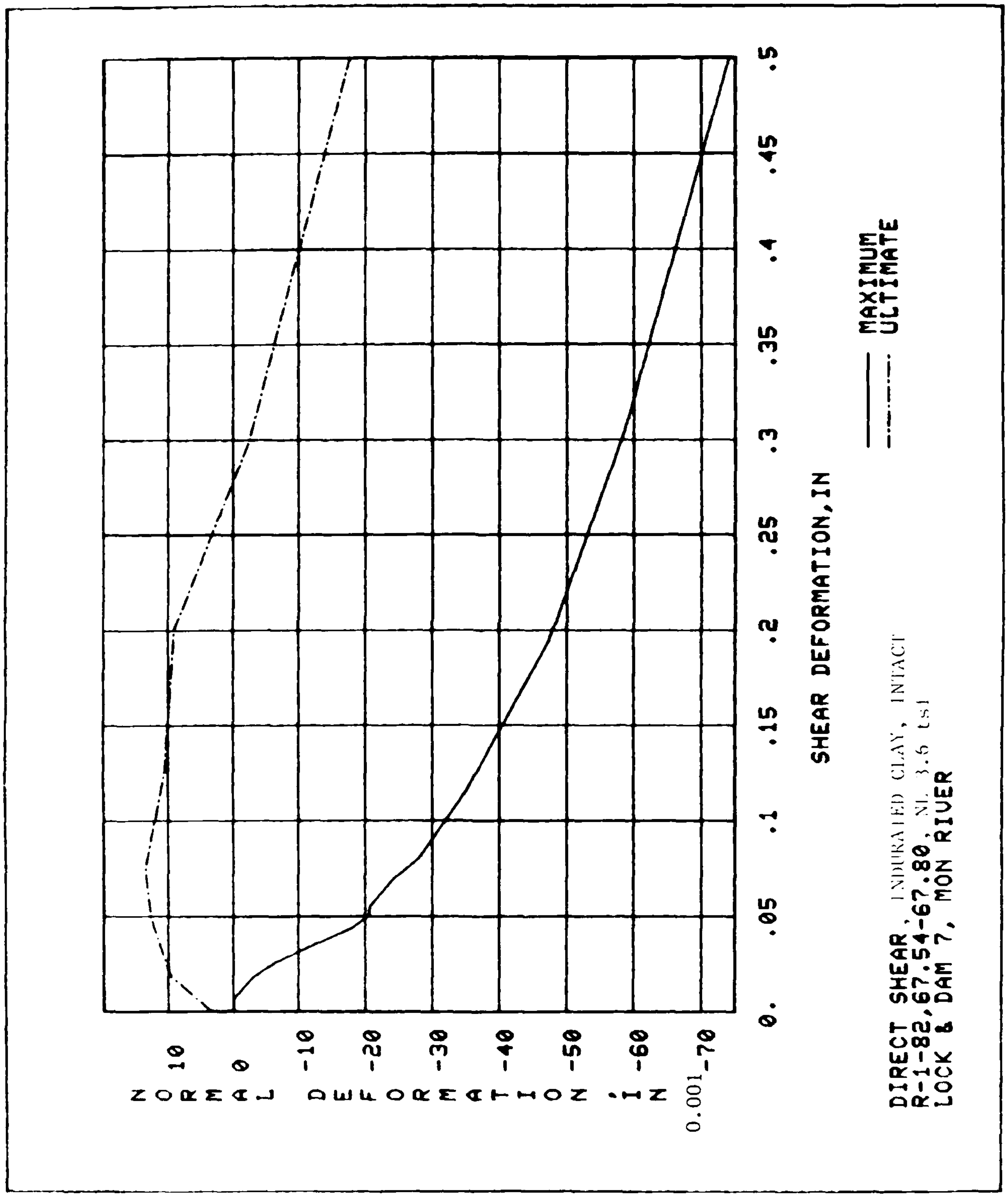
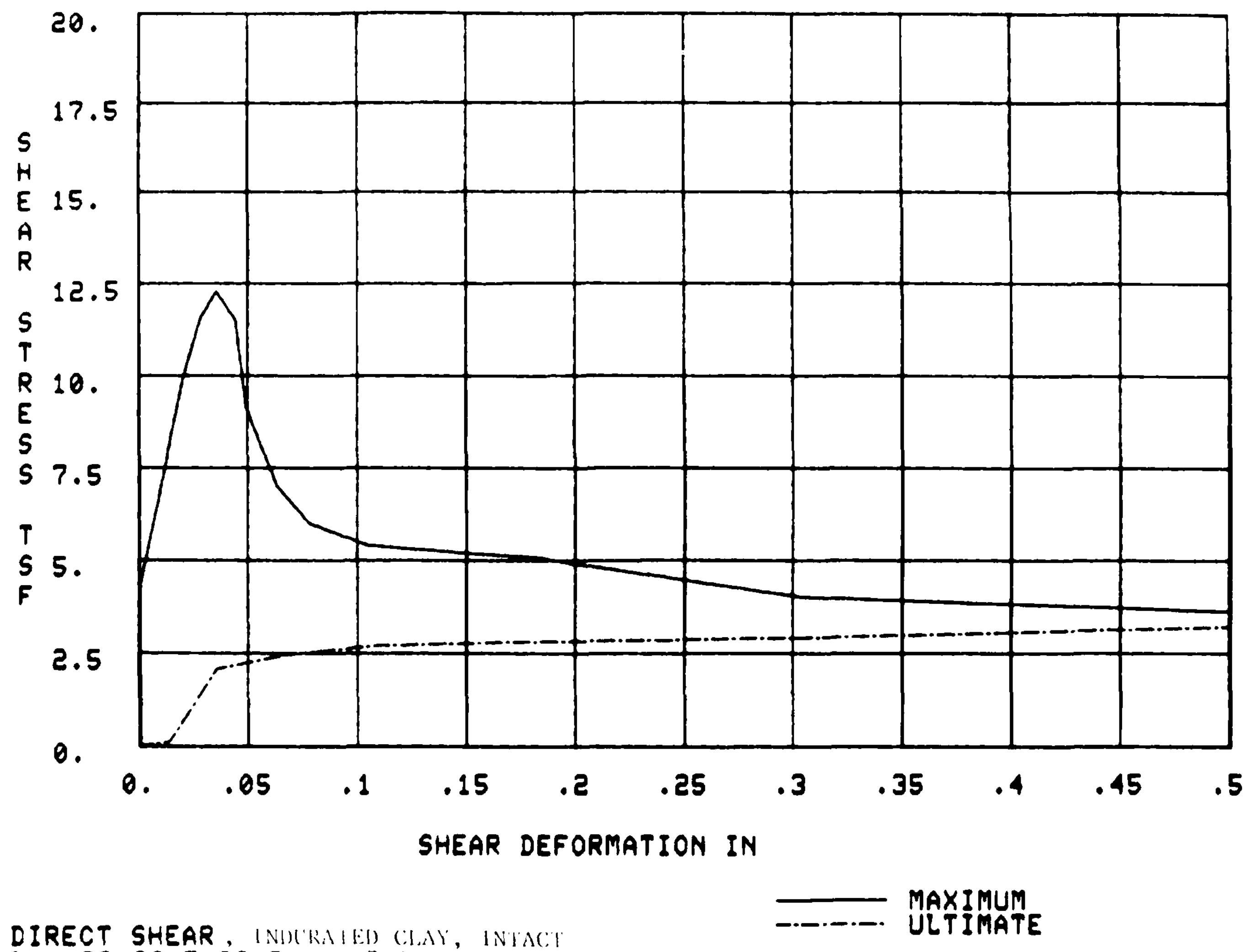
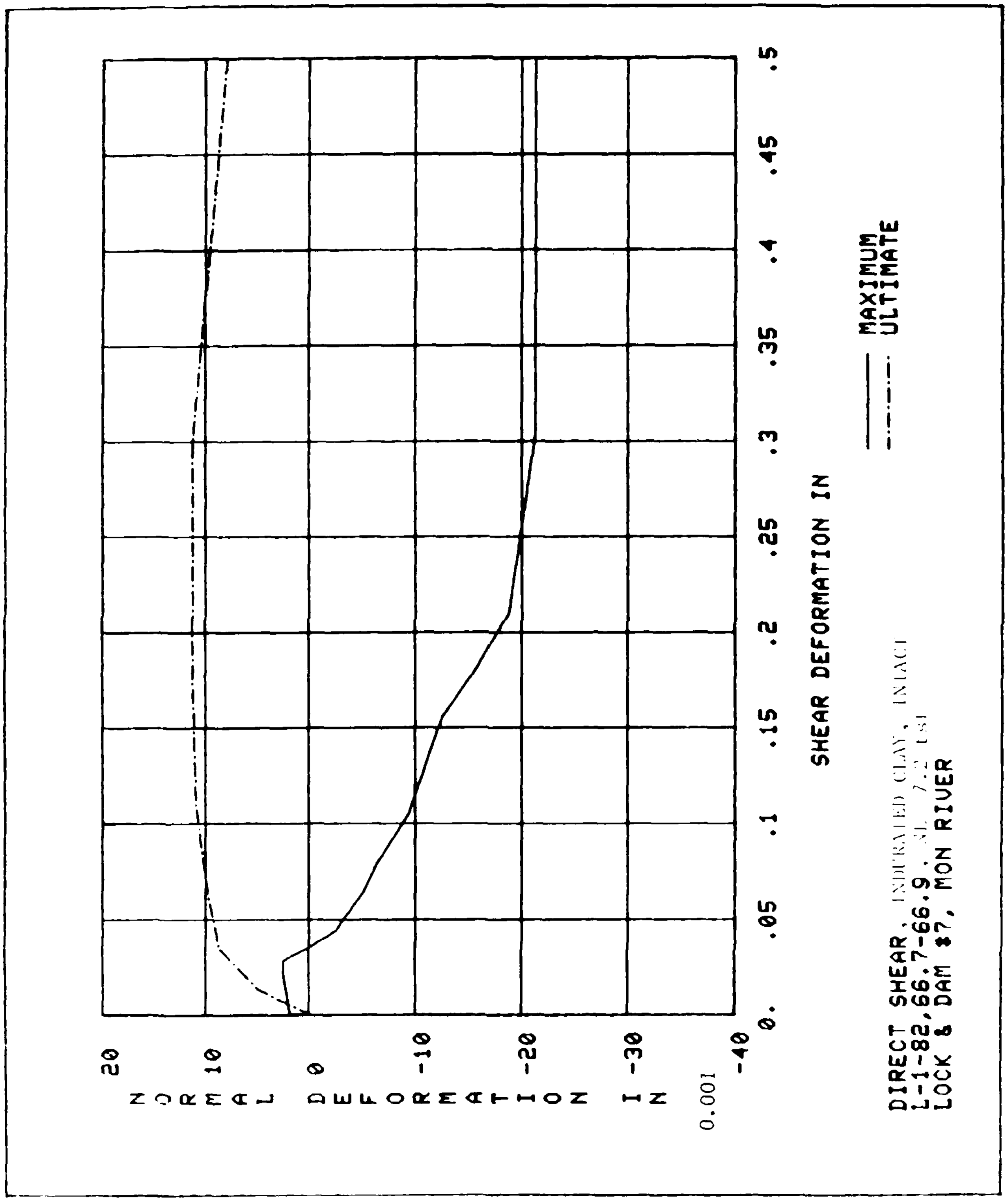
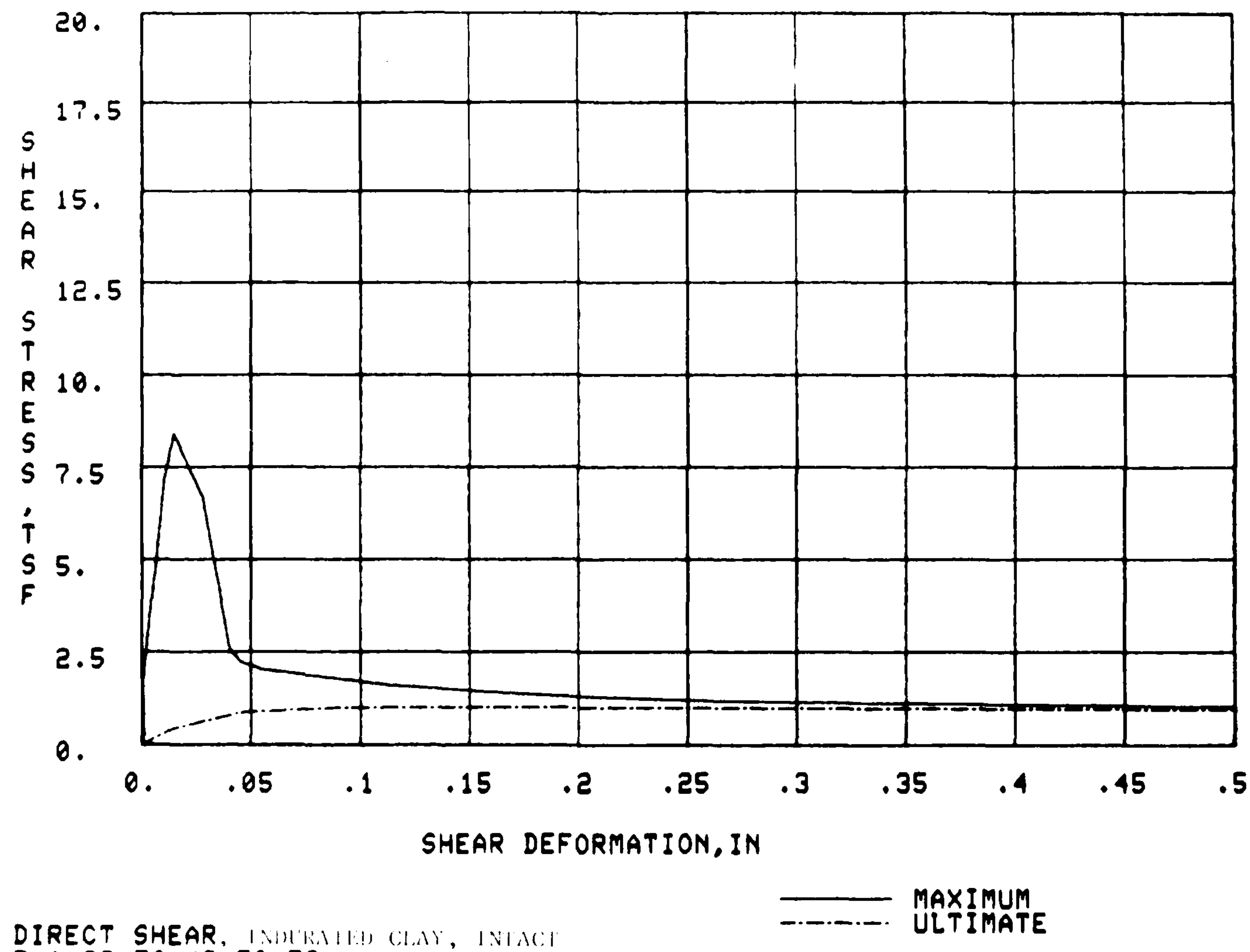

NORMAL DEFORMATION, IN 0.001
10
0
-10
-20
-30
-40
-50
-60
-70
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION, IN
MAXIMUM
ULTIMATE
DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82, 67.54-67.80, NL 3.6 tsf
LOCK & DAM 7, MON RIVER

Plate 10



DIRECT SHEAR , INDURATED CLAY, INTACT
L-1-82,66.7-66.9 , NL 7.2 tsf
LOCK & DAM #7, MON RIVER


NORMAL DEFORMATION IN 0.001
20
10
0
-10
-20
-30
-40
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
ULTIMATE
DIRECT SHEAR, INDURATED CLAY, INTACT
L-1-82,66.7-66.9, SL 7.2 tsf
LOCK & DAM #7, MON RIVER



DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82, 70.42-70.73, SL 1.8 tsf
LOCK & DAM 7, MON RIVER

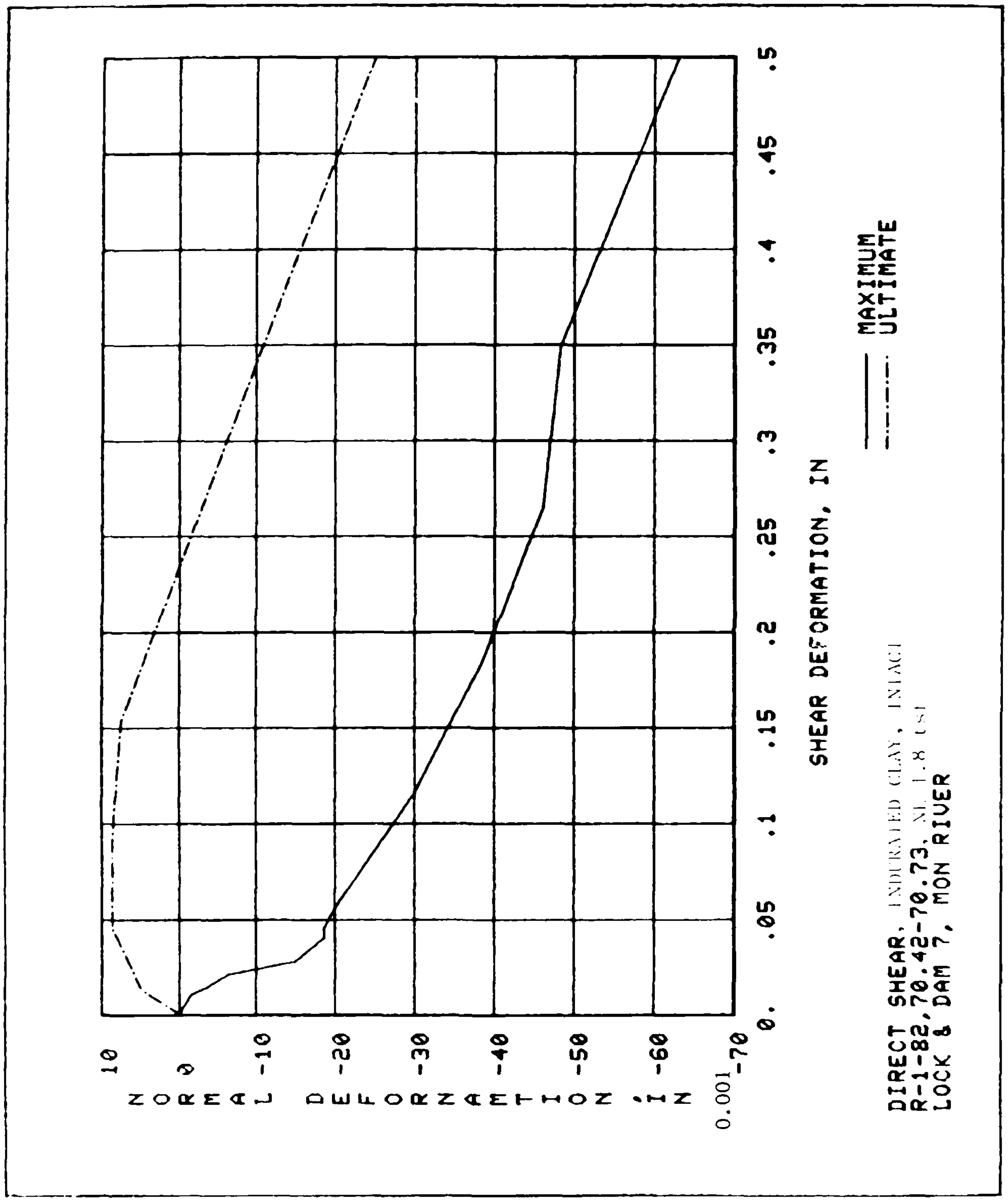
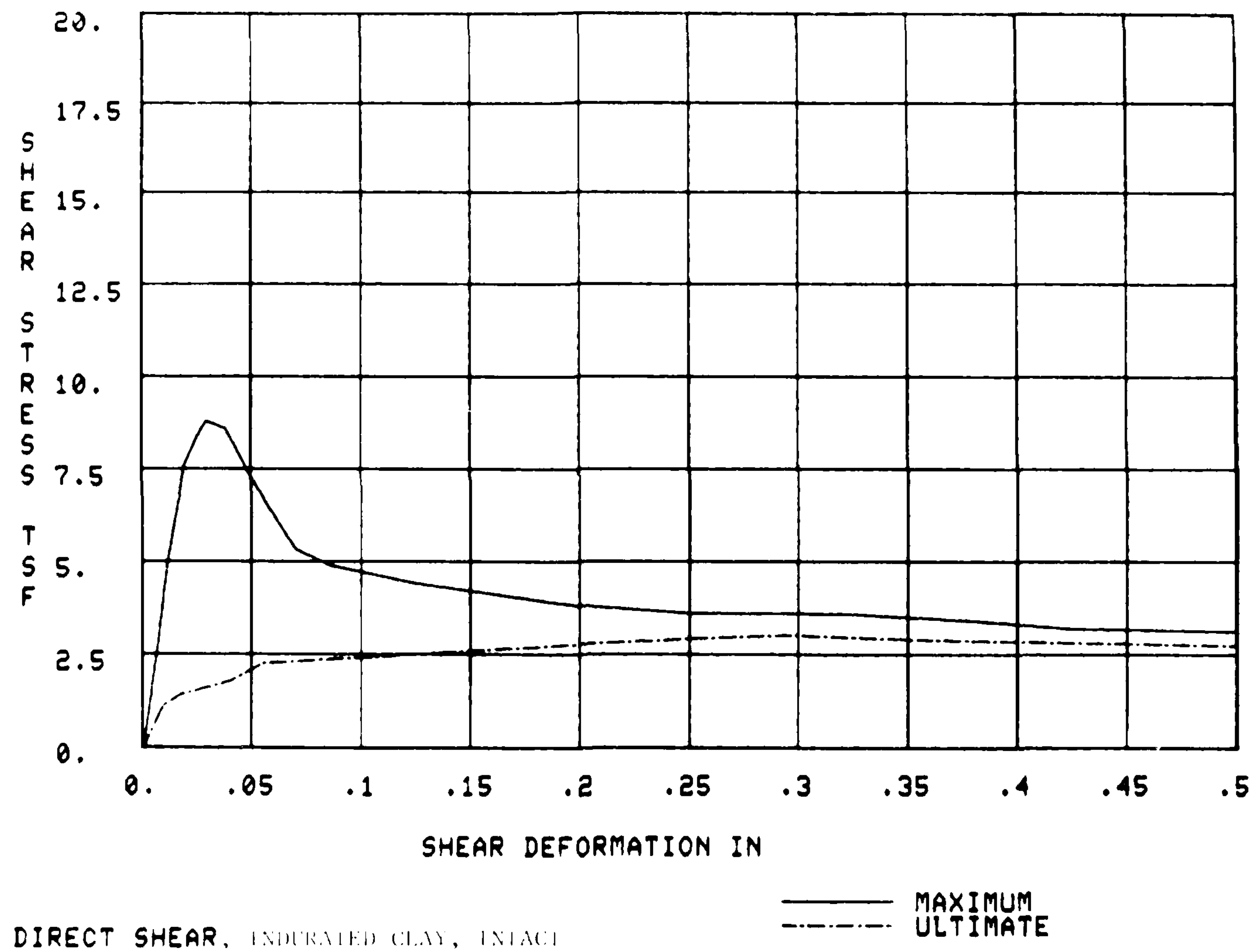
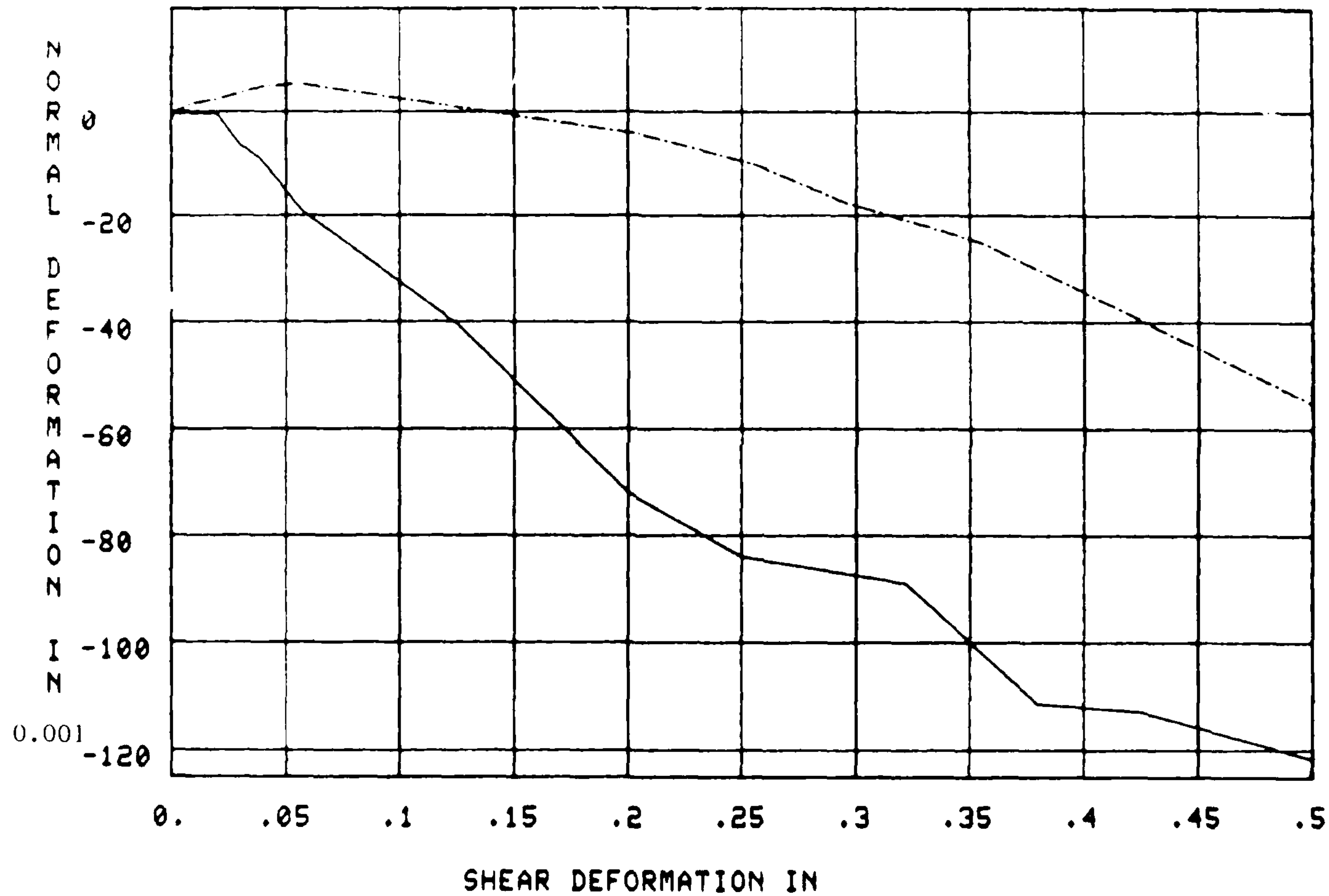

NORMAL DEFORMATION, 0.001 IN
SHEAR DEFORMATION, IN
MAXIMUM
ULTIMATE
DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82,70.42-70.73,
LOCK & DAM 7, MON RIVER

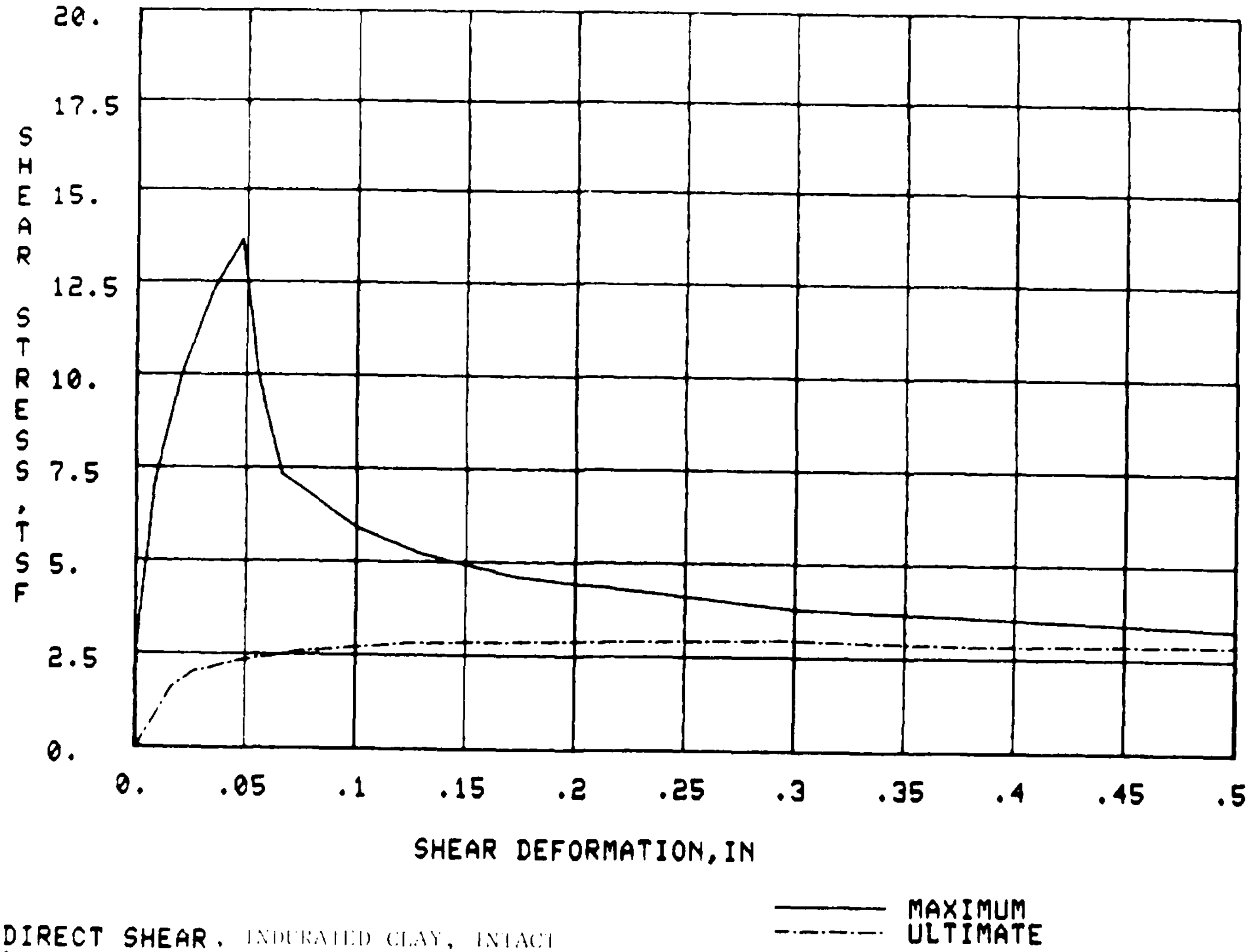
Plate 14



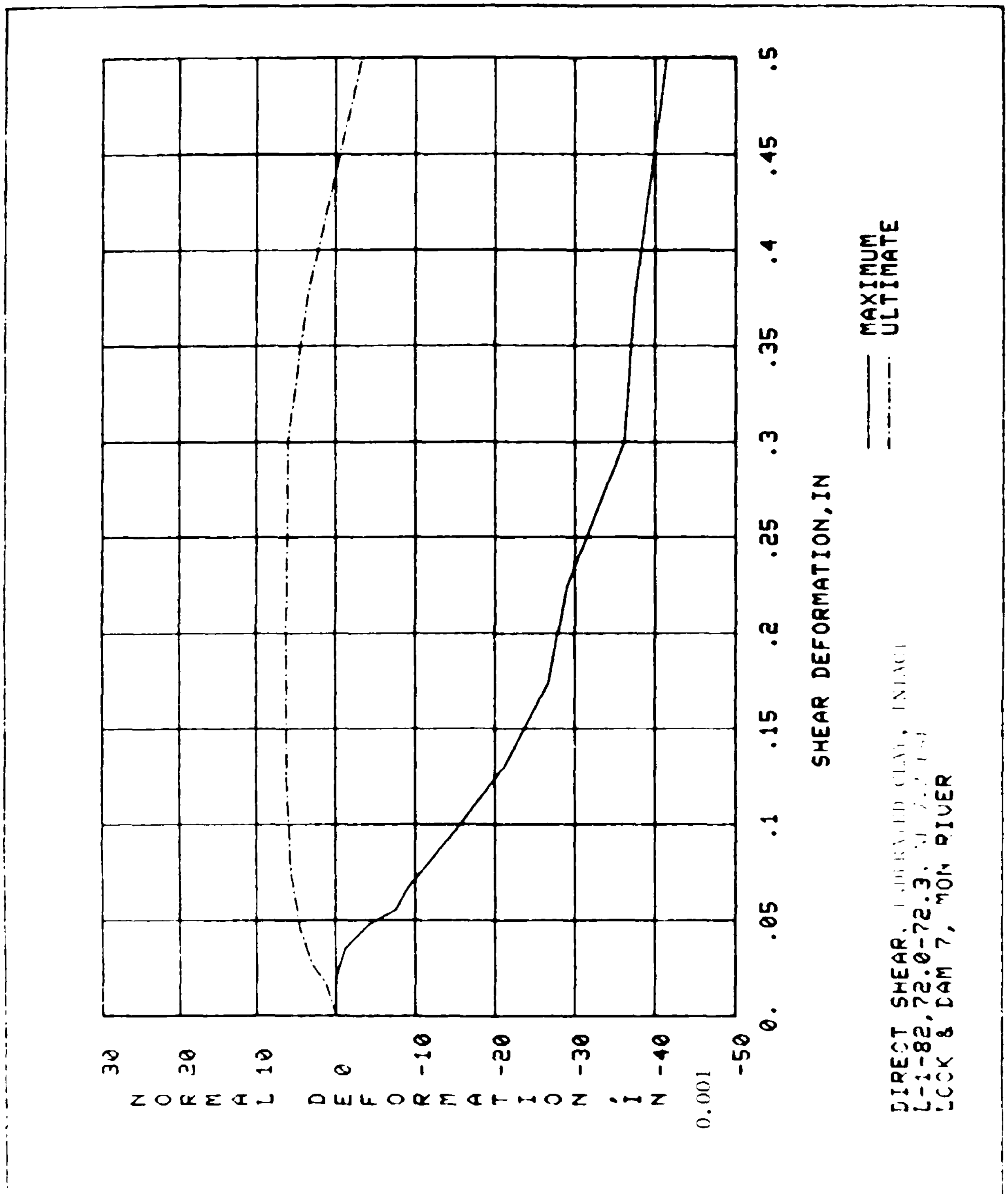
DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82,70.08-70.42, SL 3.6 tsf
LOCK & DAM #7, MON RIVER



DIRECT SHEAR, INDURATED CLAY, INTACT
R-1-82,70.08-70.42, SI 3.6 tsf
LOCK & DAM #7, MON RIVER

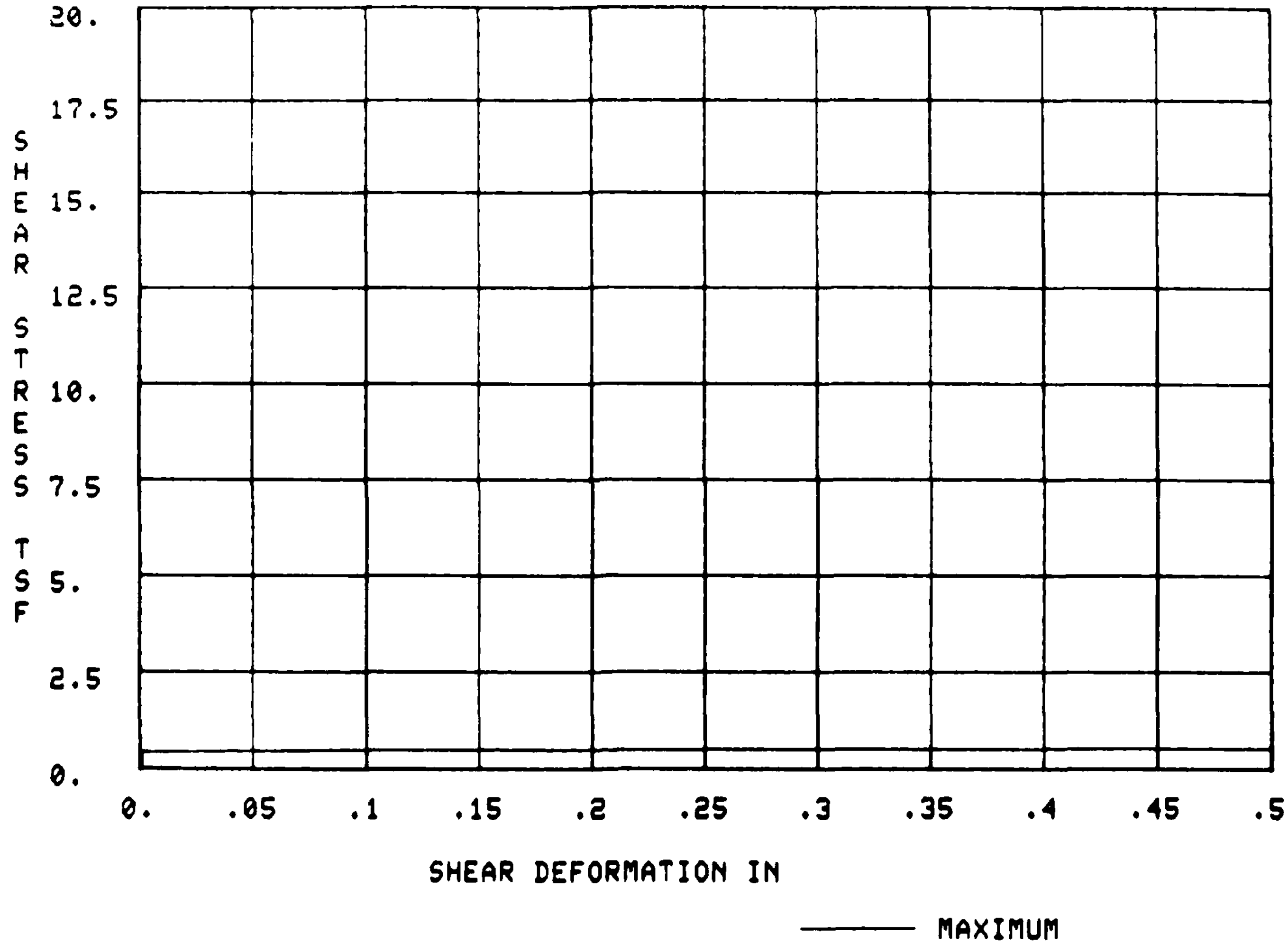


DIRECT SHEAR, INDURATED CLAY, INTACT
L-1-82, 72.0-72.3, NL 7.2 tsf
LOCK & DAM 7, MON RIVER



NORMAL DEFORMATION, IN
0.001
30
20
10
0
-10
-20
-30
-40
-50
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION,IN
MAXIMUM
ULTIMATE
DIRECT SHEAR,
L-1-82, 72.0-72.3.
LOCK & DAM 7, MON RIVER

Plate 18



DIRECT SHEAR, INDURATED CLAY PRECUT, RK ON RK
R-1-82.69.76-70.07, SL 1.5 tsf
LOCK & DAM #7, MON RIVER

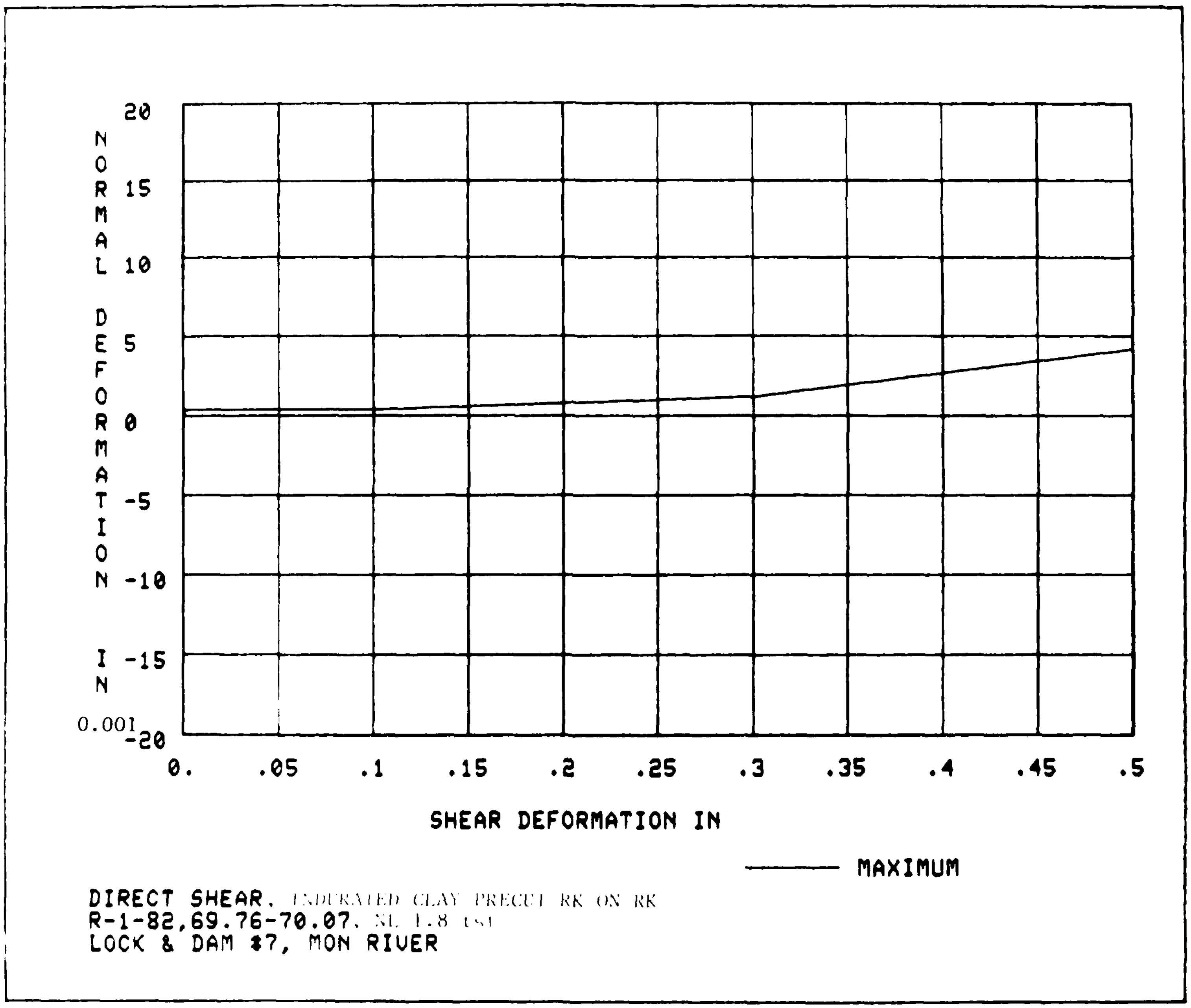
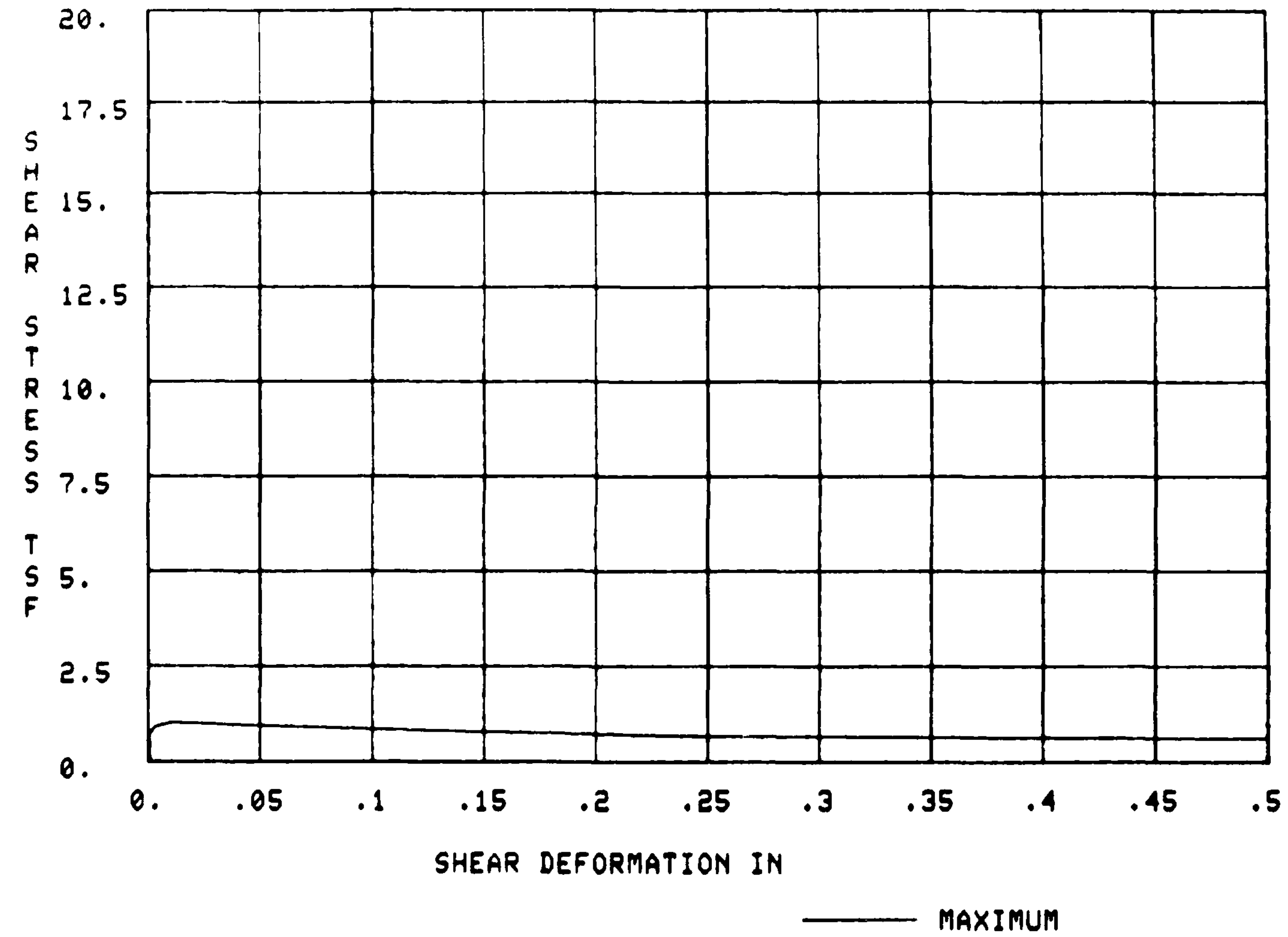
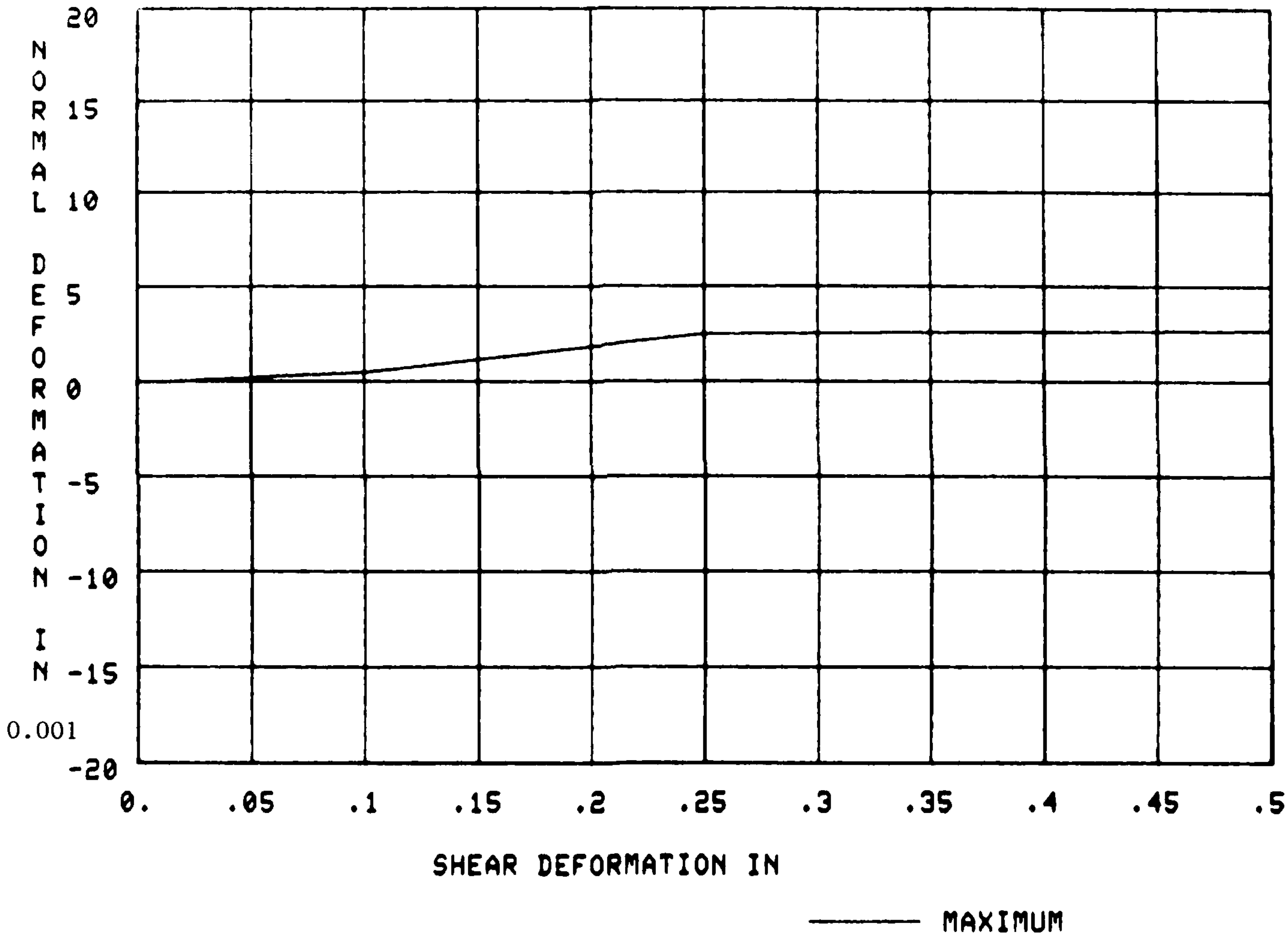

NORMAL DEFORMATION IN 0.001
20
15
10
5
0
-5
-10
-15
-20
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
DIRECT SHEAR, INDURATED CLAY PRECUT RK ON RK
R-1-82,69.76-70.07, NL 1.8 tsf
LOCK & DAM #7, MON RIVER

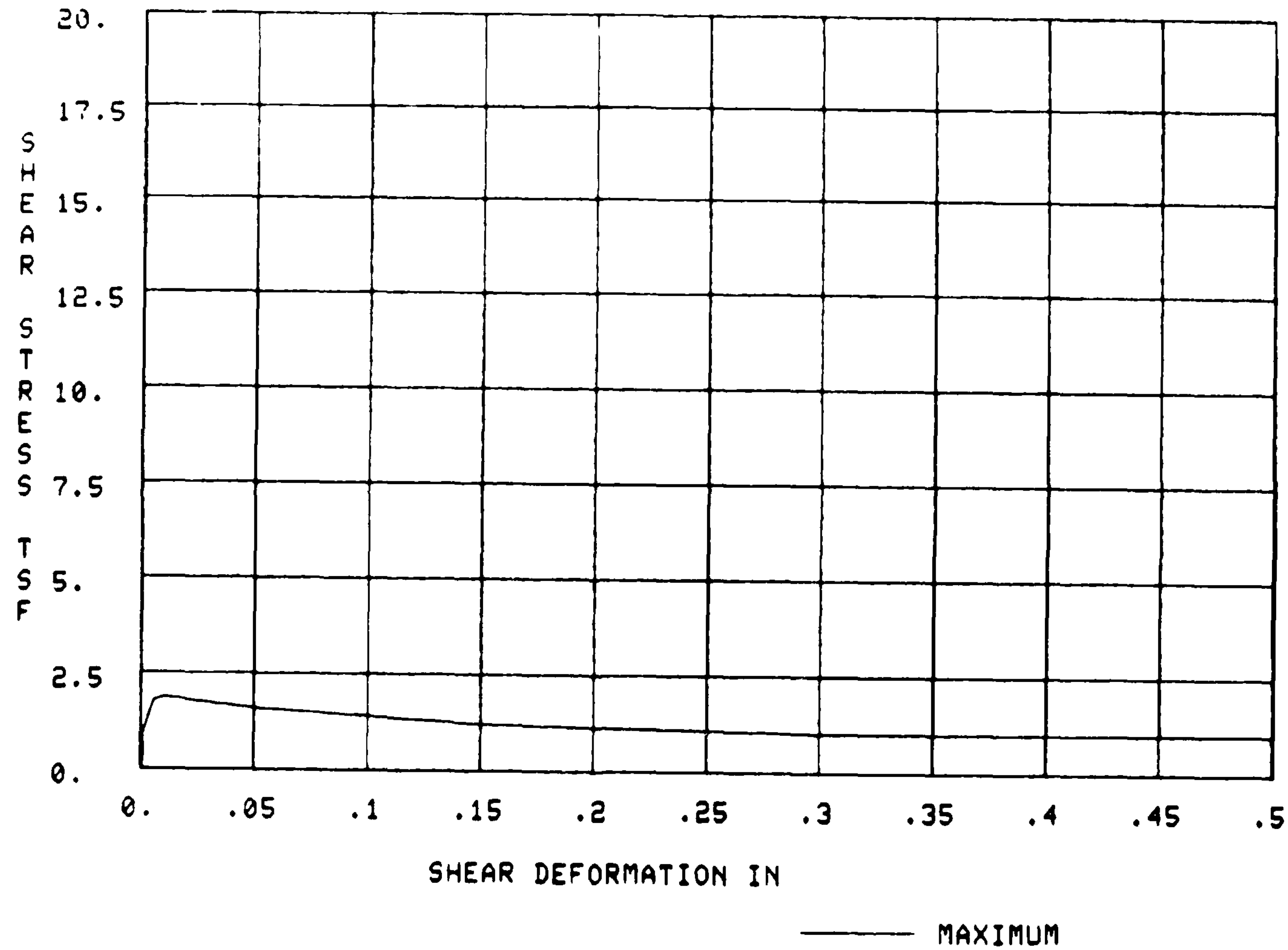
Plate 20



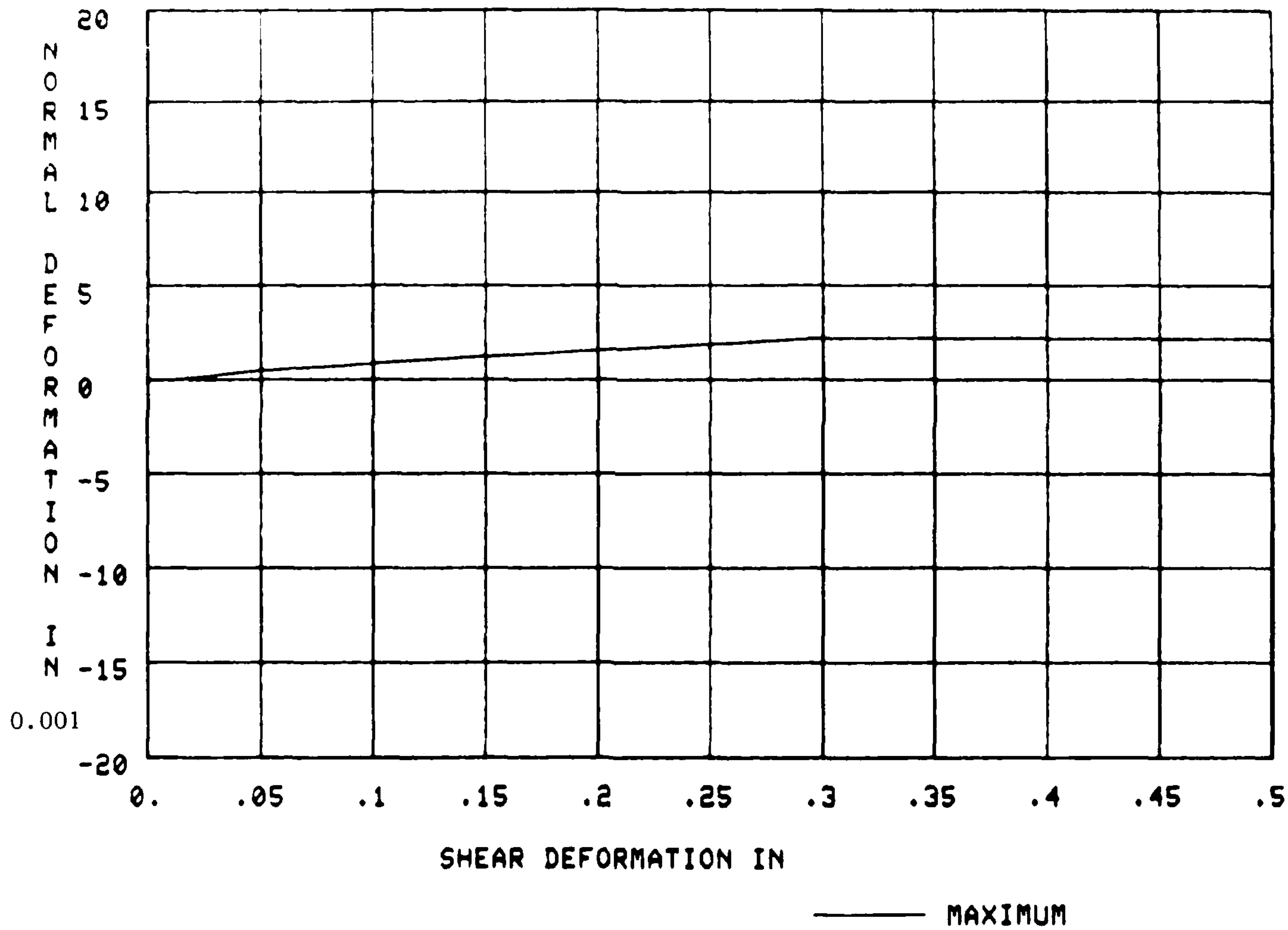
DIRECT SHEAR, INDURATED CLAY PRECUT RK ON RK
R-1-82,69.45-69.86, NL 3.6 tsf
LOCK & DAM #7, MON RIVER



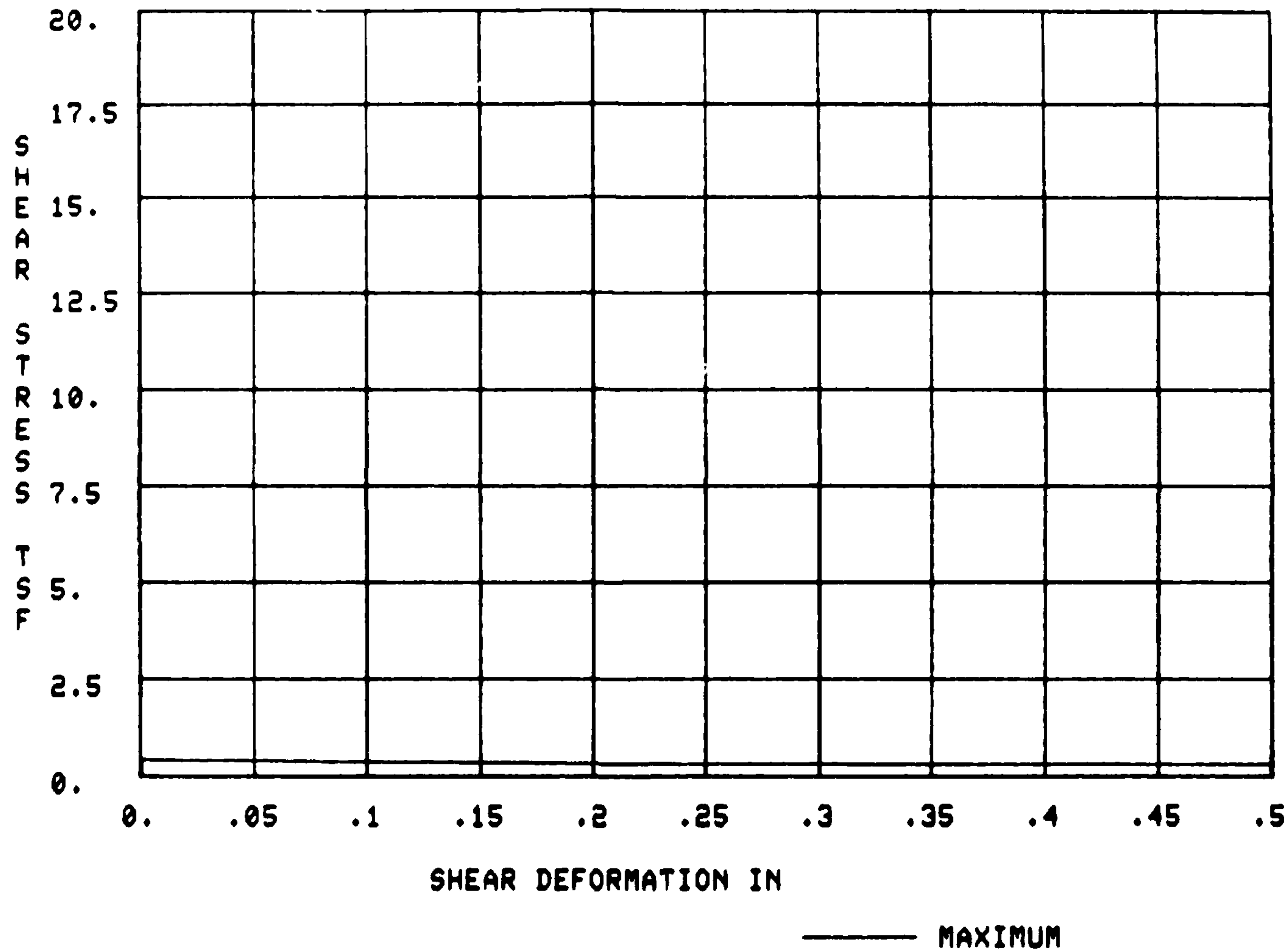
DIRECT SHEAR, INDURATED CLAY PRECUT RK ON RK
R-1-82,69.45-69.86, NL 3.6 TSF
LOCK & DAM #7, MON RIVER



DIRECT SHEAR, INDURATED CLAY PRECUT RK ON RK
R-1-82,67.8-68.1, NL 7.2 tsf
LOCK & DAM #7, MON RIVER



DIRECT SHEAR, INDURATED CLAY PRECUT RK ON RK
R-1-82,67.8-68.1, σ_n 7.2 tsf
LOCK & DAM #7, MON RIVER



DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,68.0-68.2, NL 1.8 TSF
LOCK & DAM #7 MON RIVER

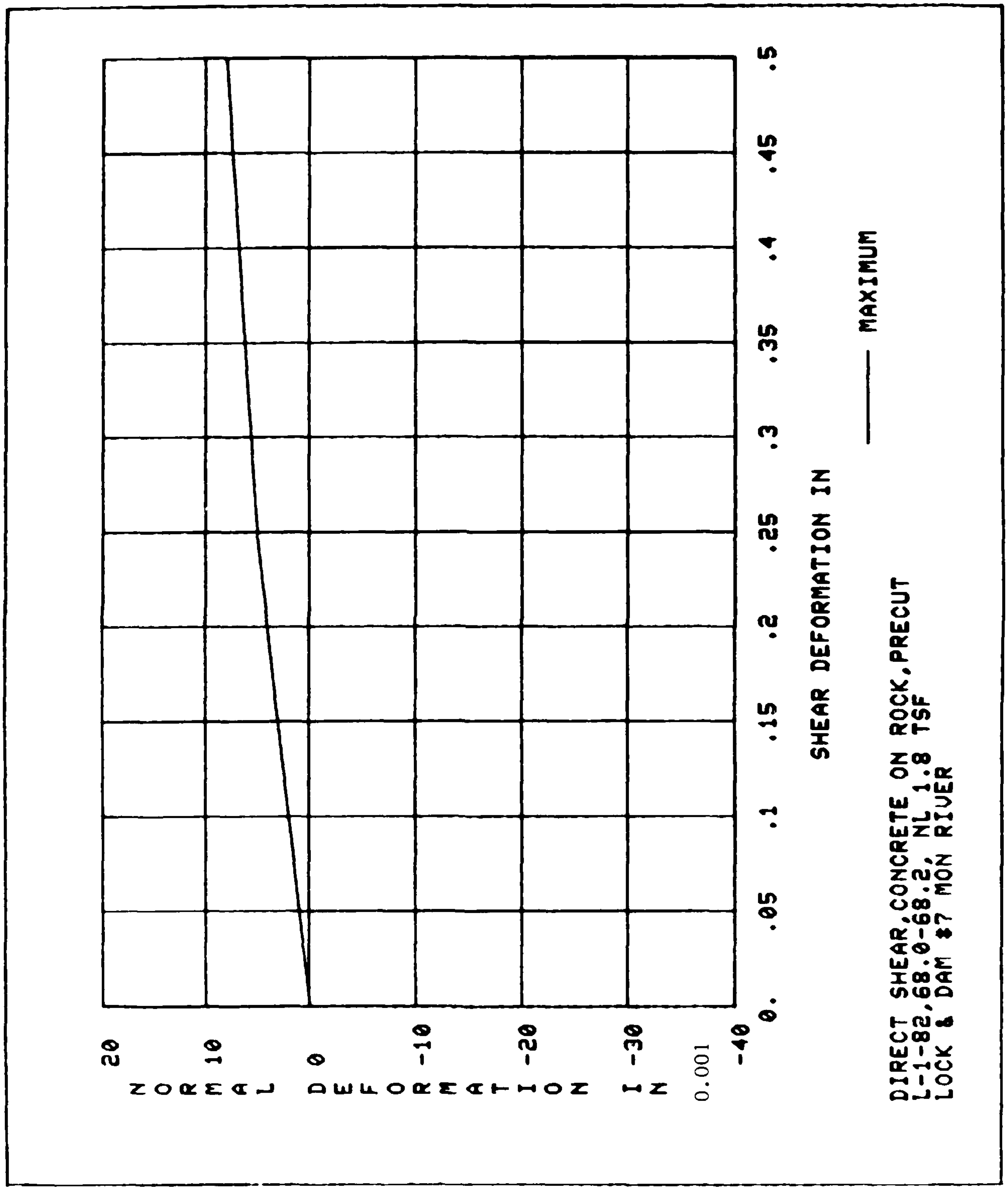
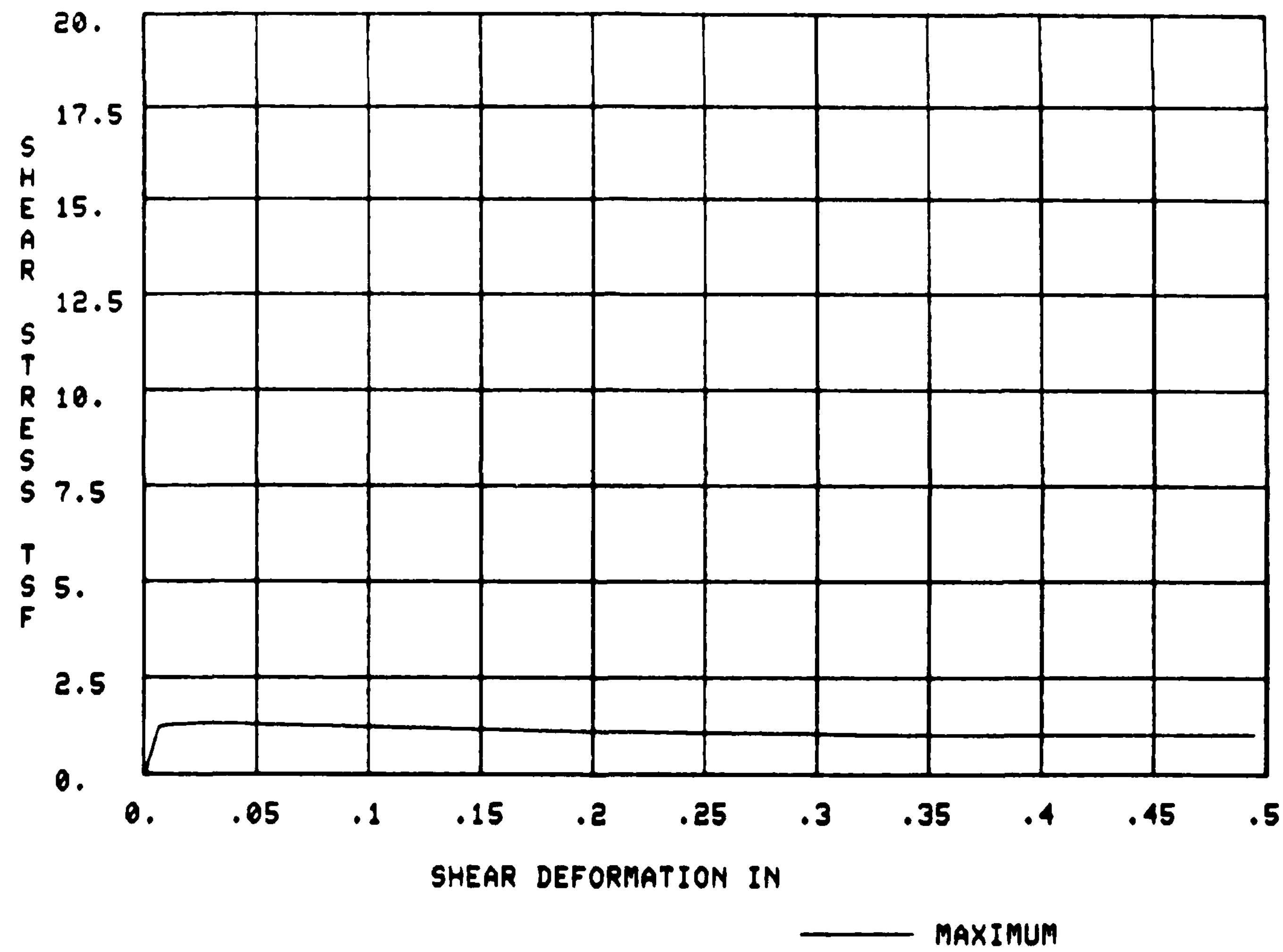
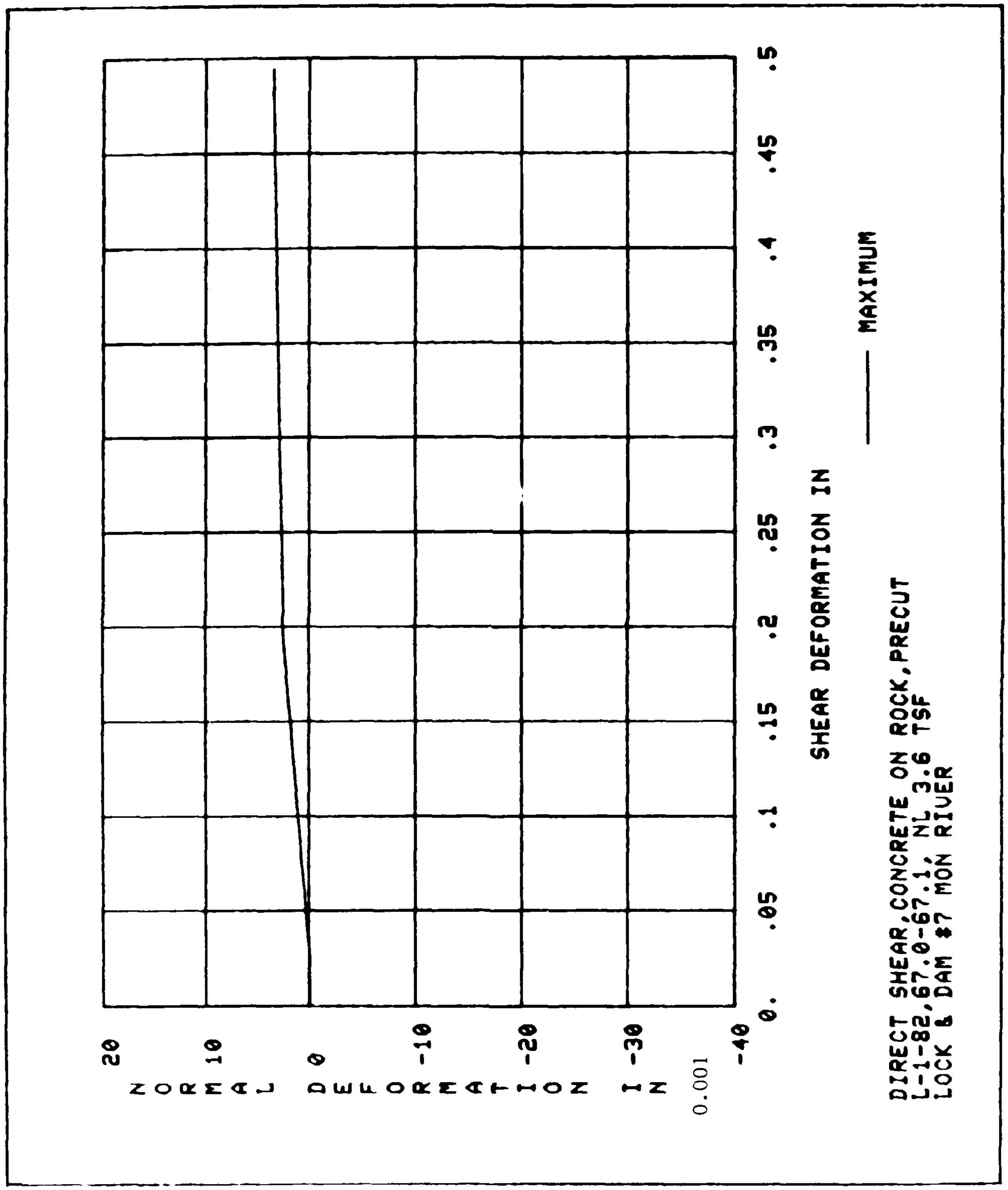

NORMAL DEFORMATION IN 0.001
20
10
0
-10
-20
-30
-40
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,68.0-68.2, NL 1.8 TSF
LOCK & DAM #7 MON RIVER

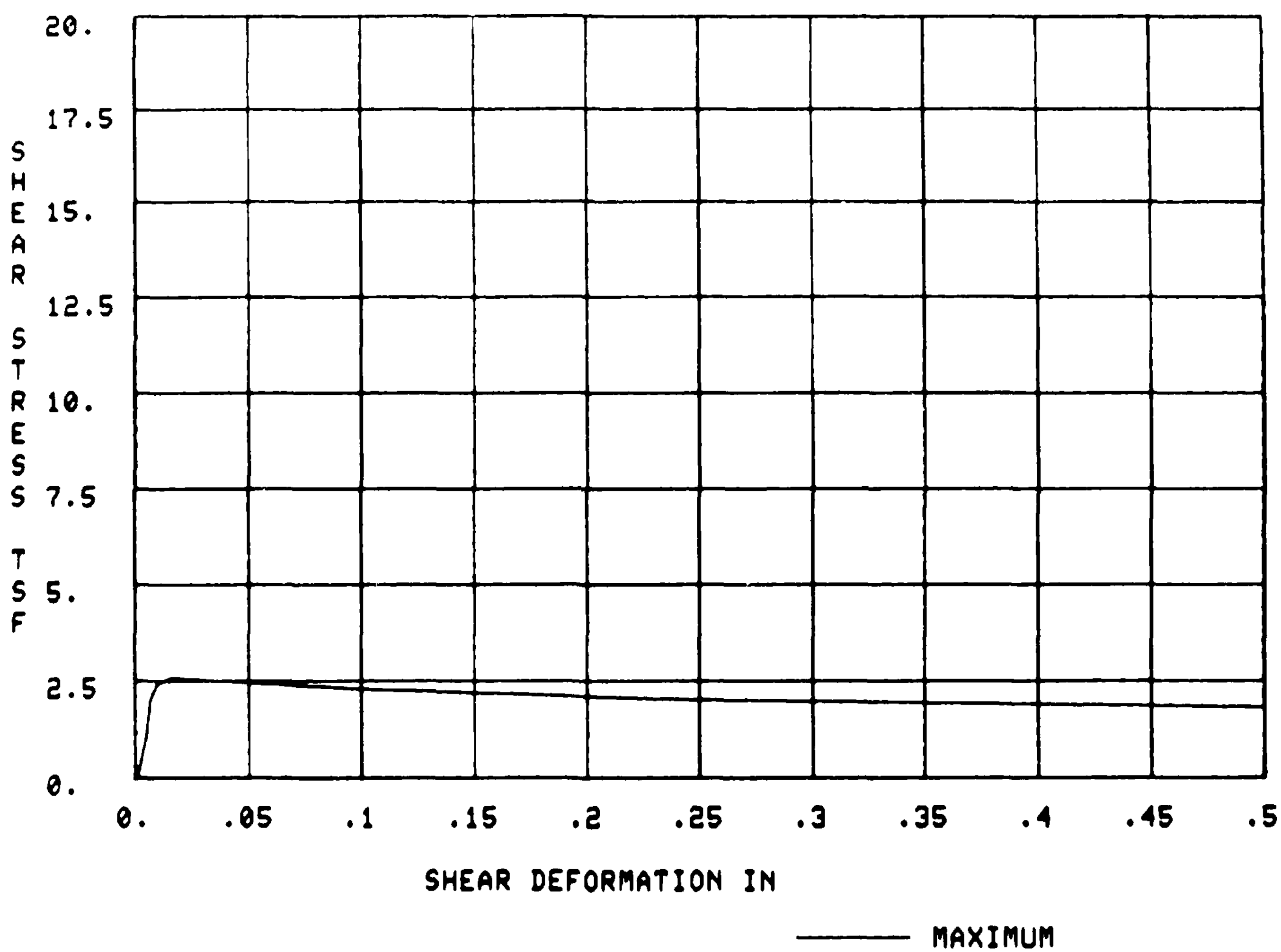
Plate 26



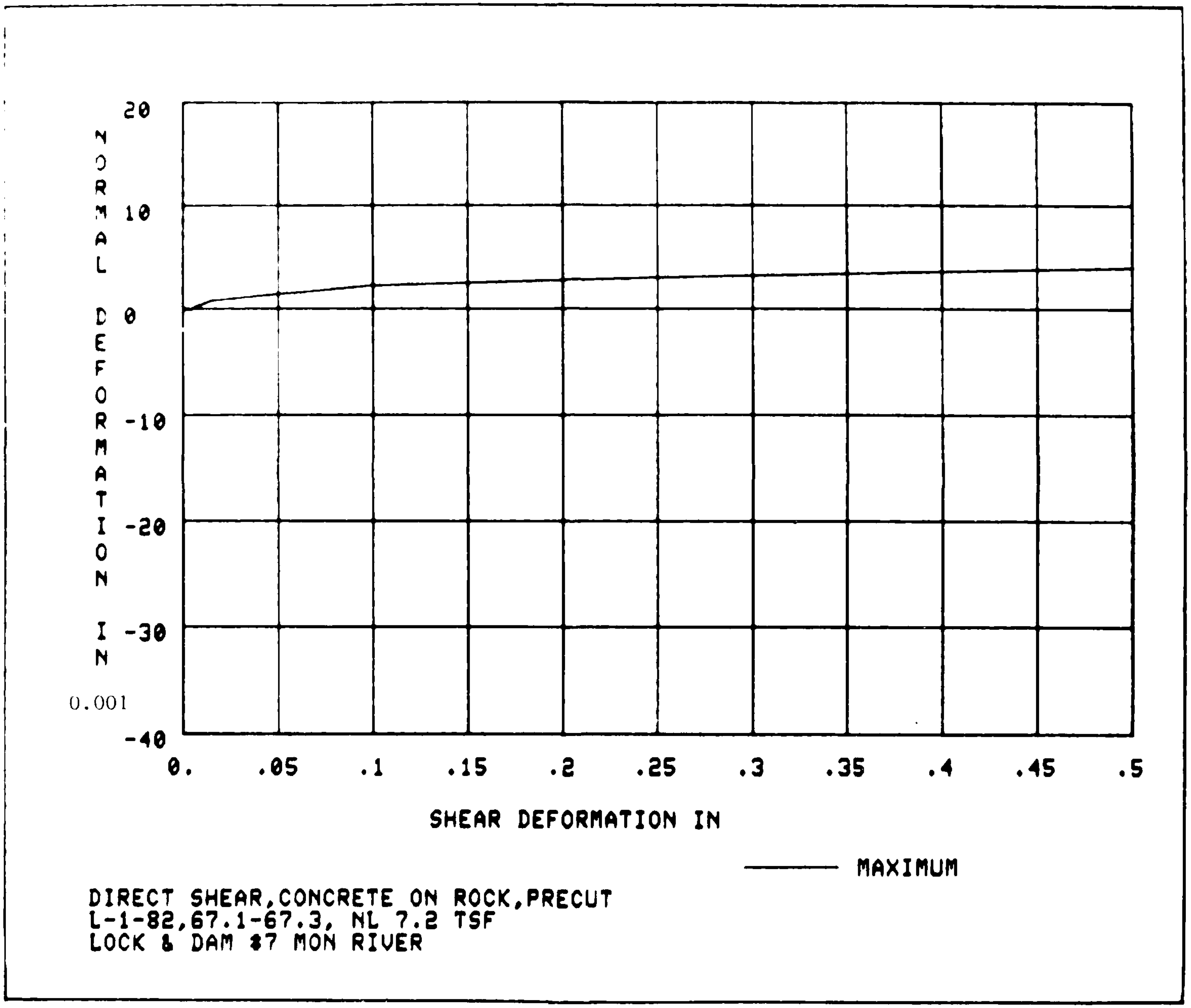
DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,67.0-67.1, NL 3.6 TSF
LOCK & DAM #7 MON RIVER



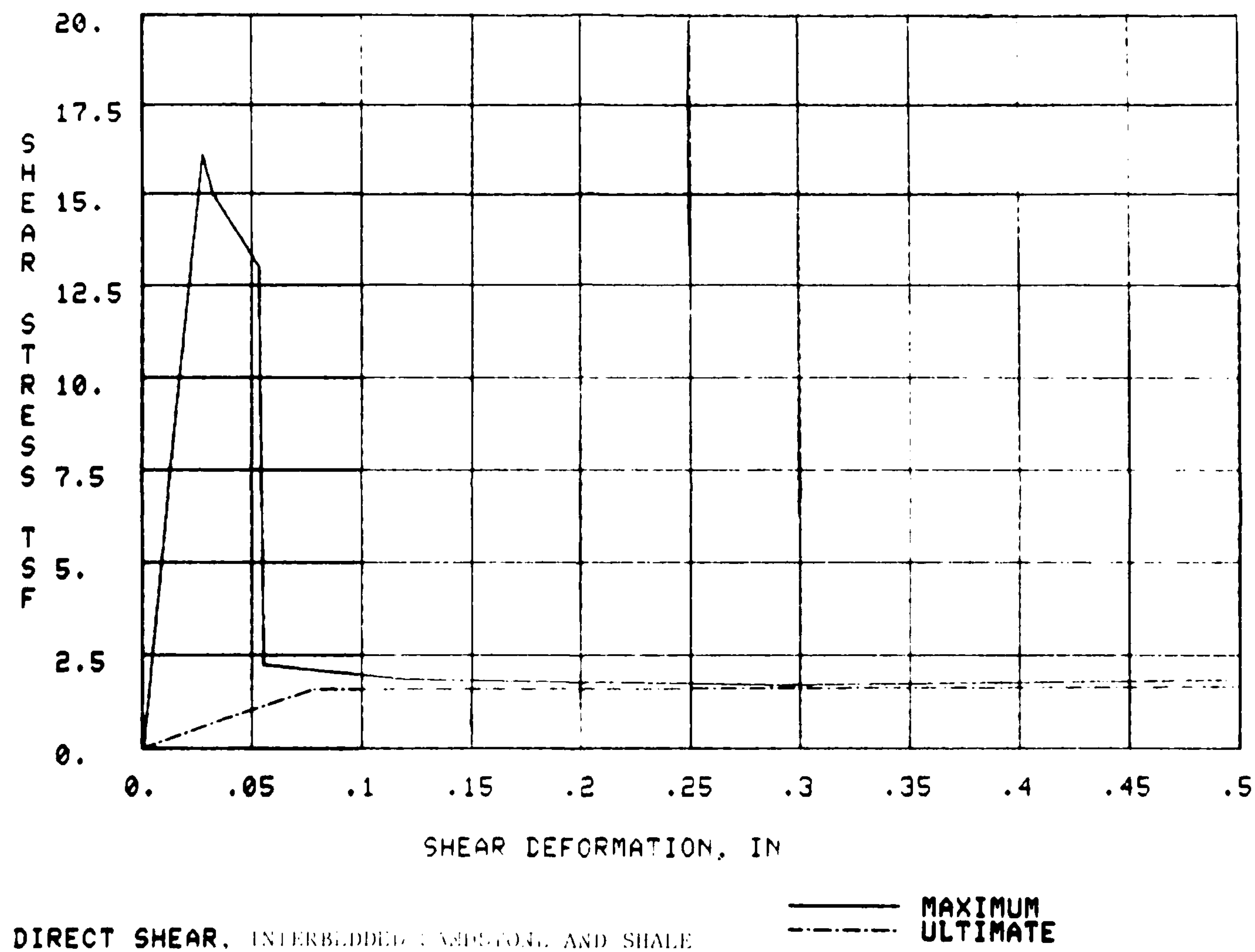
SHEAR DEFORMATION IN
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
NORMAL DEFORMATION IN 0.001
20
10
0
-10
-20
-30
-40
MAXIMUM
DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,67.0-67.1, NL 3.6 TSF
LOCK & DAM #7 MON RIVER



DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,67.1-67.3, NL 7.2 TSF
LOCK & DAM #7 MON RIVER



NORMAL DEFORMATION IN 0.001
20
10
0
-10
-20
-30
-40
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION IN
MAXIMUM
DIRECT SHEAR,CONCRETE ON ROCK,PRECUT
L-1-82,67.1-67.3, NL 7.2 TSF
LOCK & DAM #7 MON RIVER



DIRECT SHEAR, INTERBEDDED SANDSTONE AND SHALE
R-1-82,53.0-53.42, [illegible]
LOCK & DAM 7, MON RIVER

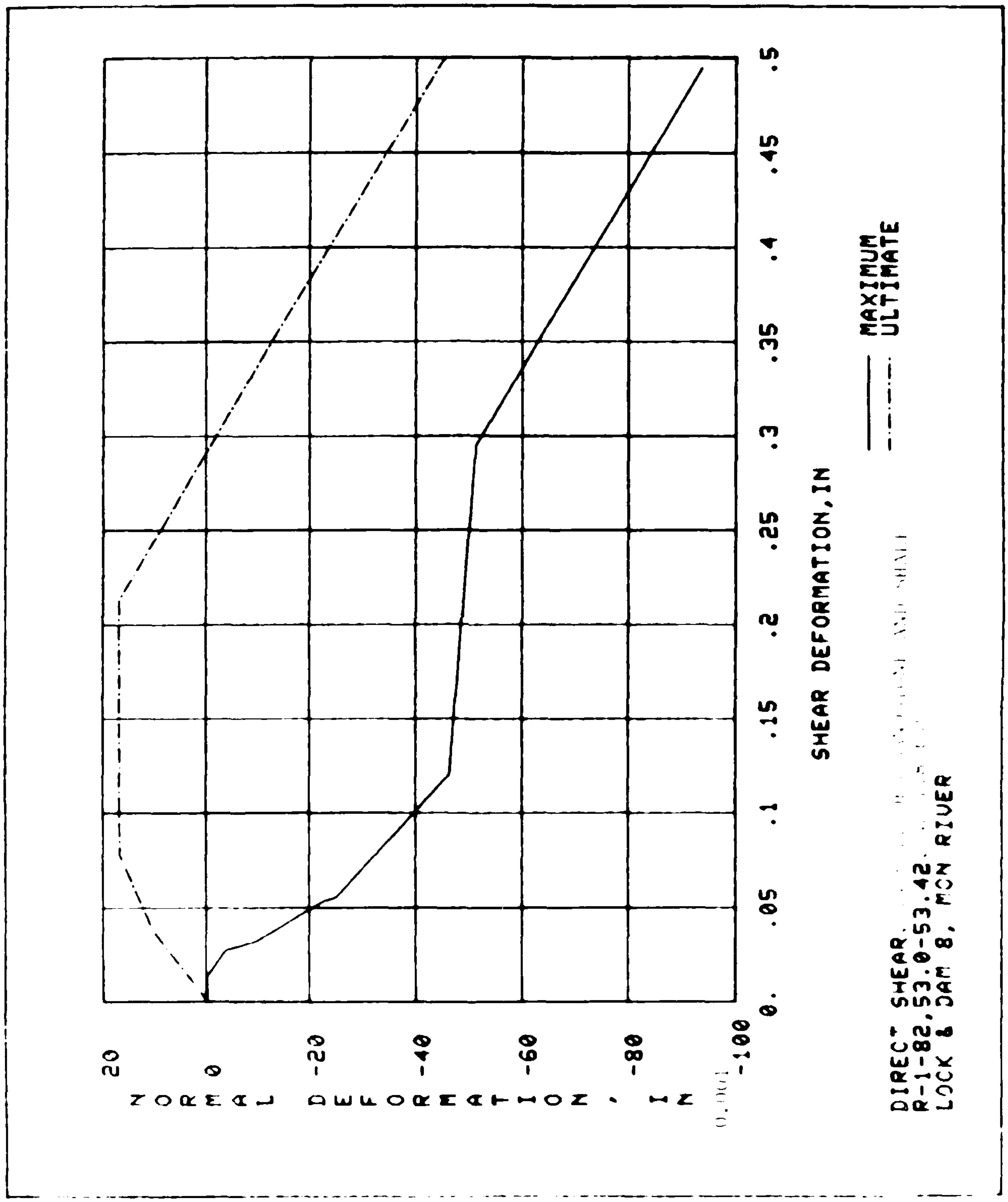
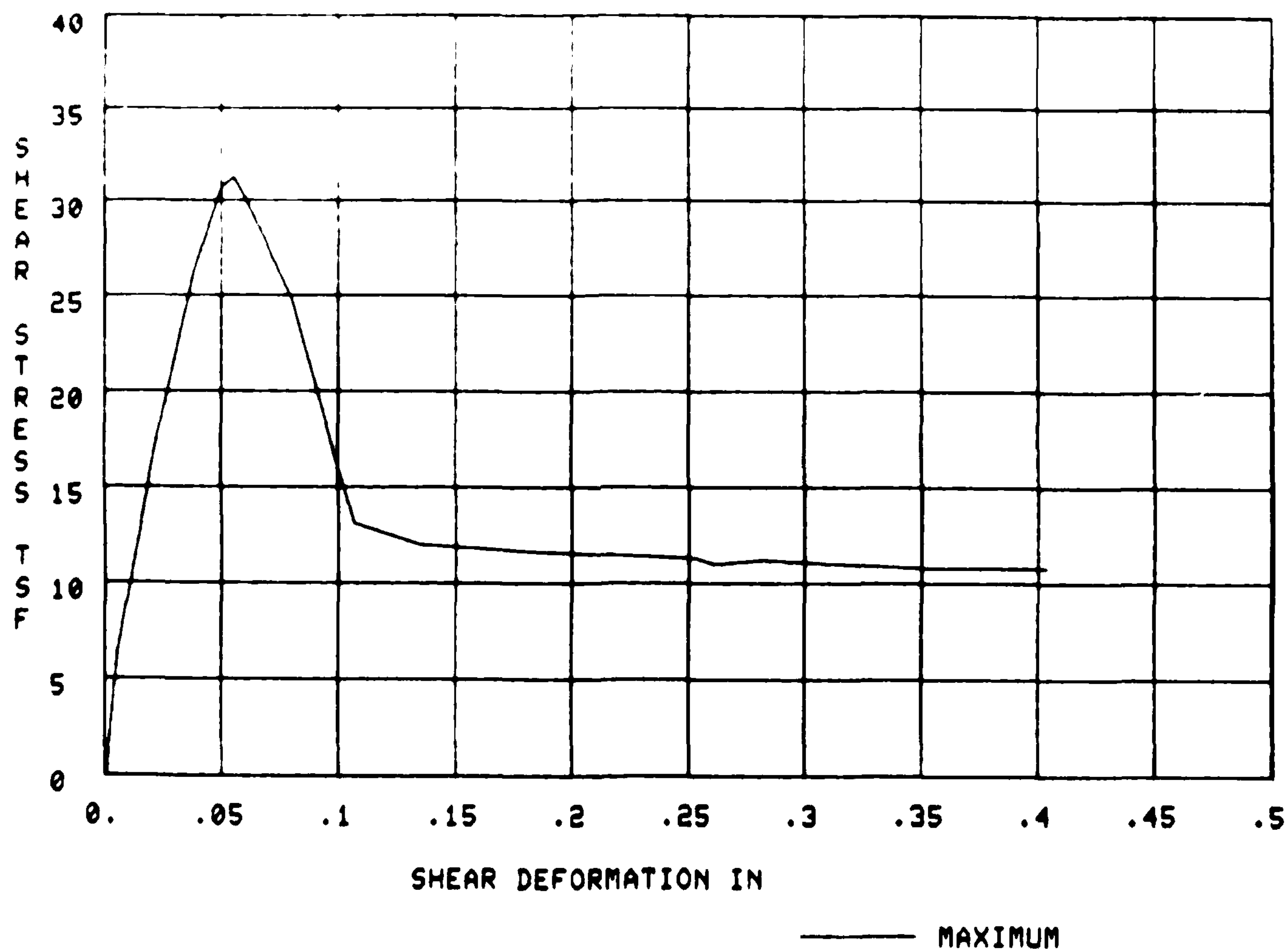
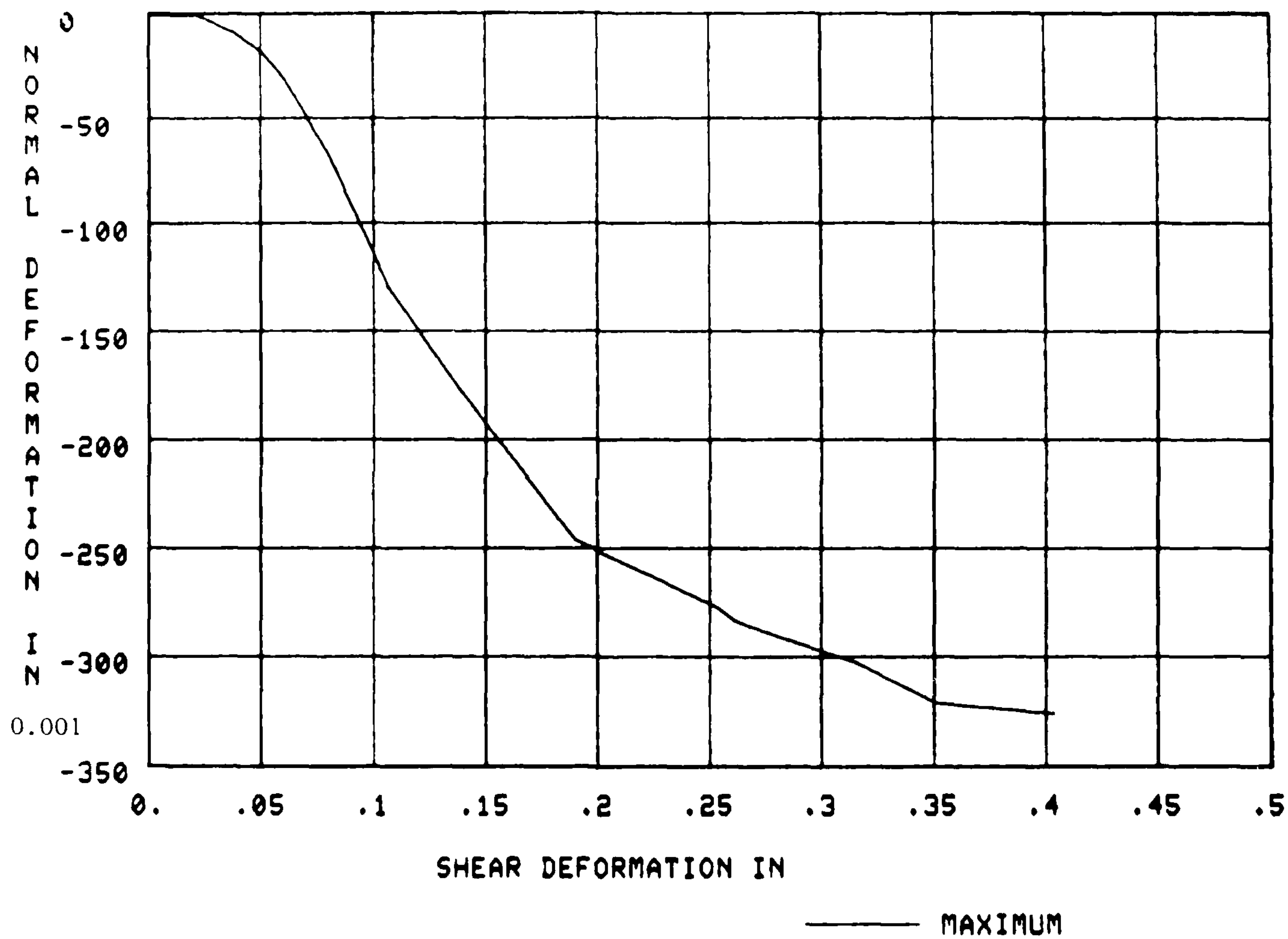

NORMAL DEFORMATION, IN
20
0
-20
-40
-60
-80
-100
0.
.05
.1
.15
.2
.25
.3
.35
.4
.45
.5
SHEAR DEFORMATION, IN
MAXIMUM
ULTIMATE
DIRECT SHEAR
R-1-82, 53.0-53.42
LOCK & DAM 8, MON RIVER

Plate 32

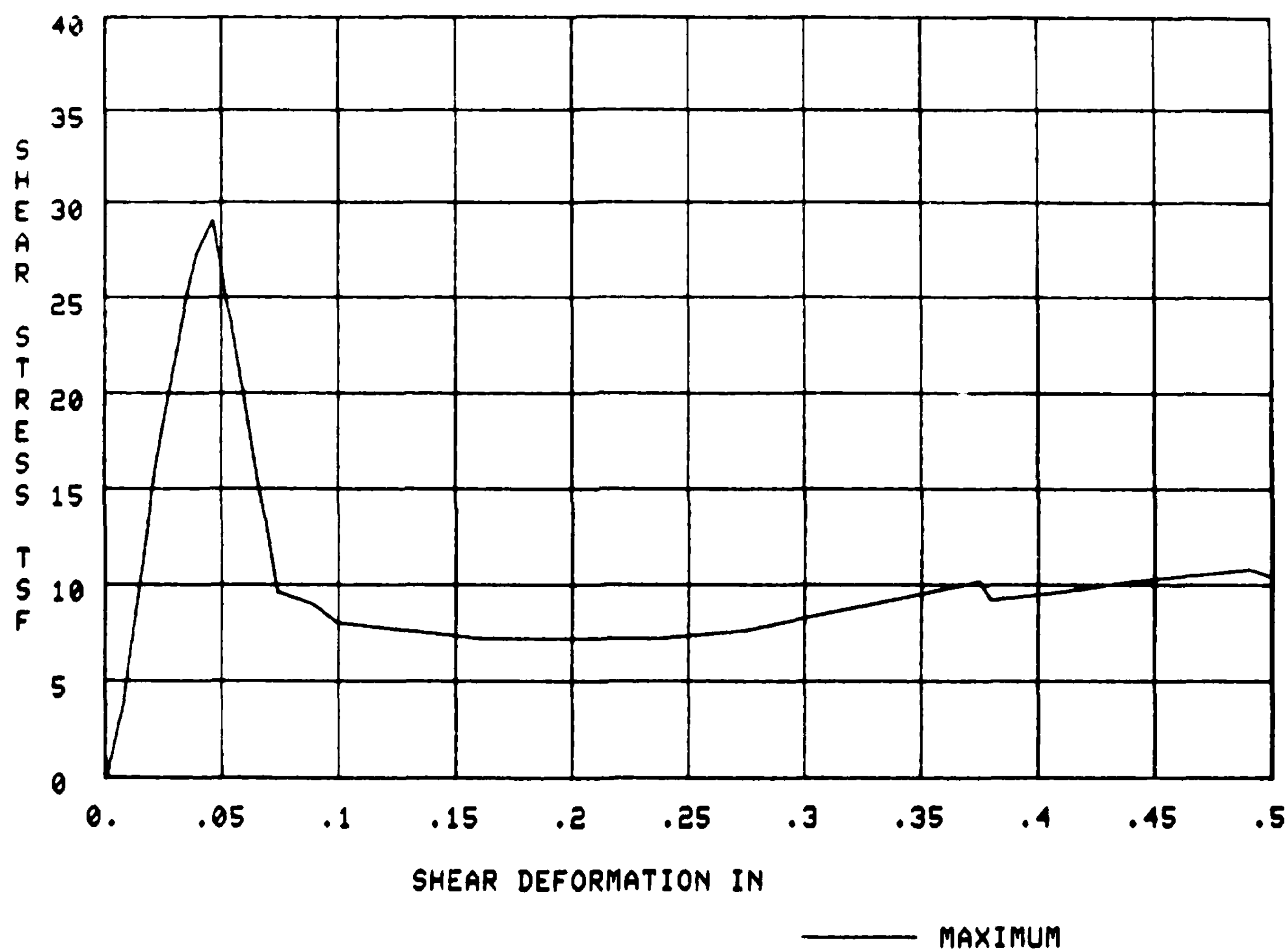


DIRECT SHEAR, INTERBEDDED SANDSTONE AND SHALE
R-1-82,53.43-53.84, NL 3.6 tsf
LOCK & DAM #7, MON RIVER

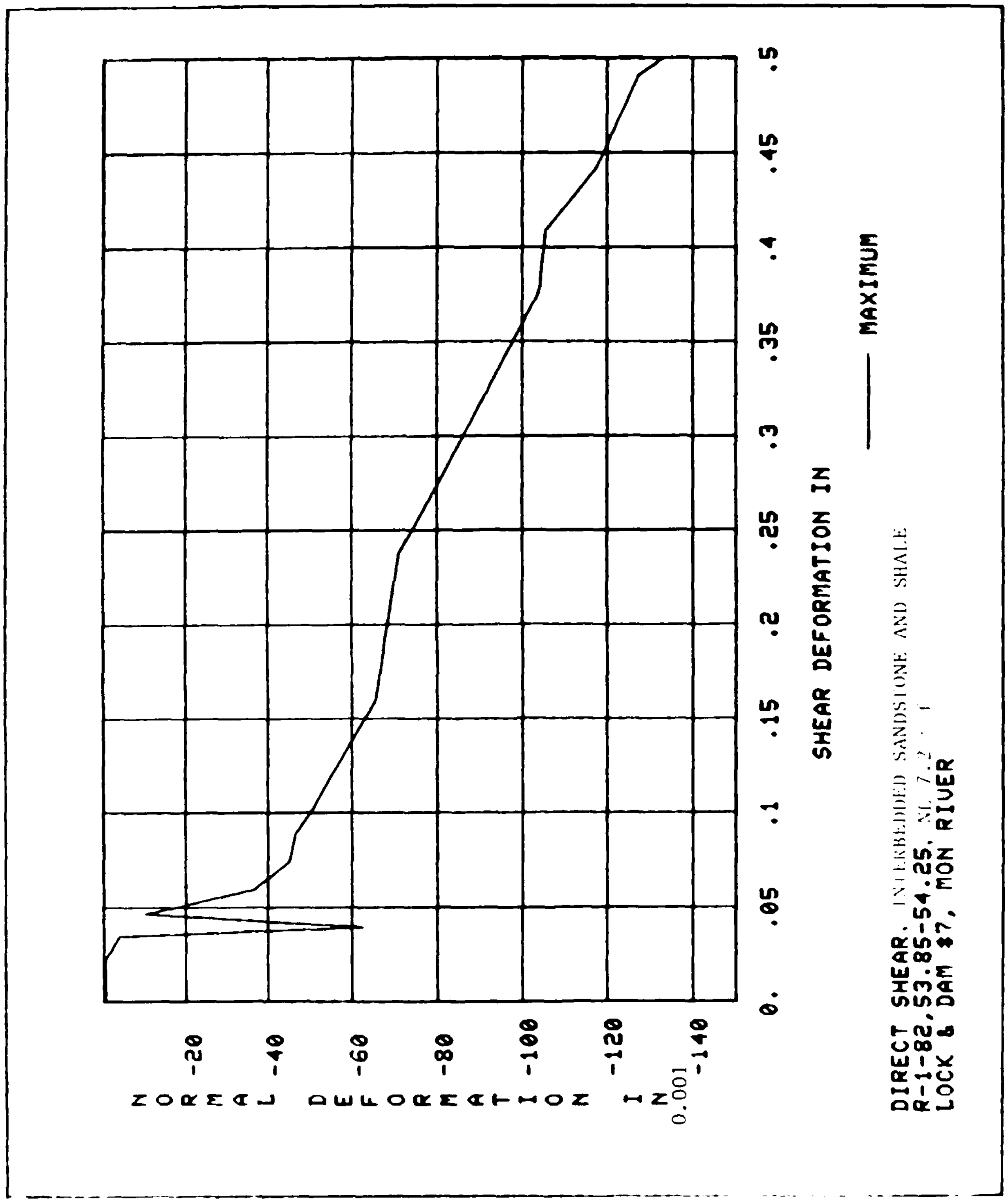


DIRECT SHEAR, INTERBEDDED SANDSTONE AND SHALE
R-1-82,53.43-53.84, NL 3.6 ksi
LOCK & DAM #7, MON RIVER

Plate 34



DIRECT SHEAR, INTERBEDDED SANDSTONE AND SHALE
R-1-82,53.85-54.25, NL 7.2 tsf
LOCK & DAM #7, MON RIVER

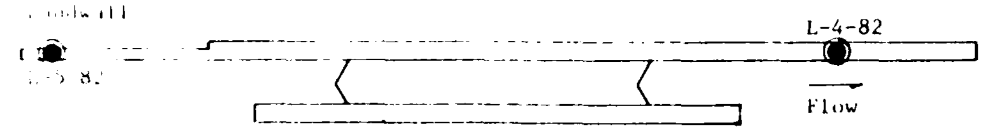


SHEAR DEFORMATION IN
NORMAL DEFORMATION IN 0.001
0. .05 .1 .15 .2 .25 .3 .35 .4 .45 .5
-20 -40 -60 -80 -100 -120 -140
MAXIMUM
DIRECT SHEAR. INTERBEDDED SANDSTONE AND SHALE
R-1-82,53.85-54.25,
LOCK & DAM #7, MON RIVER

Plate 36

CONCRETE PROPERTIES

Lock #7 Mon River



Vertical Boring Locations

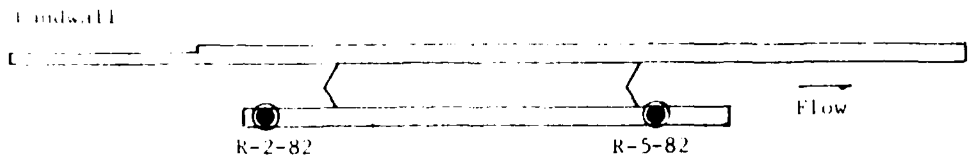
Boring No. [illegible]

Elev	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
[illegible]			Frost damage 0.0-1.6'					
	1.0							
	2.0							
	3.0							

Boring No. L-5-82

Elev	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
788.0			Frost damage 0.9-1.1'					
	1.0		Overlay 0.0-0.9'					
	2.0		Alkali-silica gel in voids and on breaks					
785.0	3.0							
	4.0							

CONCRETE PROPERTIES
Lock #7 Mon River



Vertical Boring Locations

No Scale

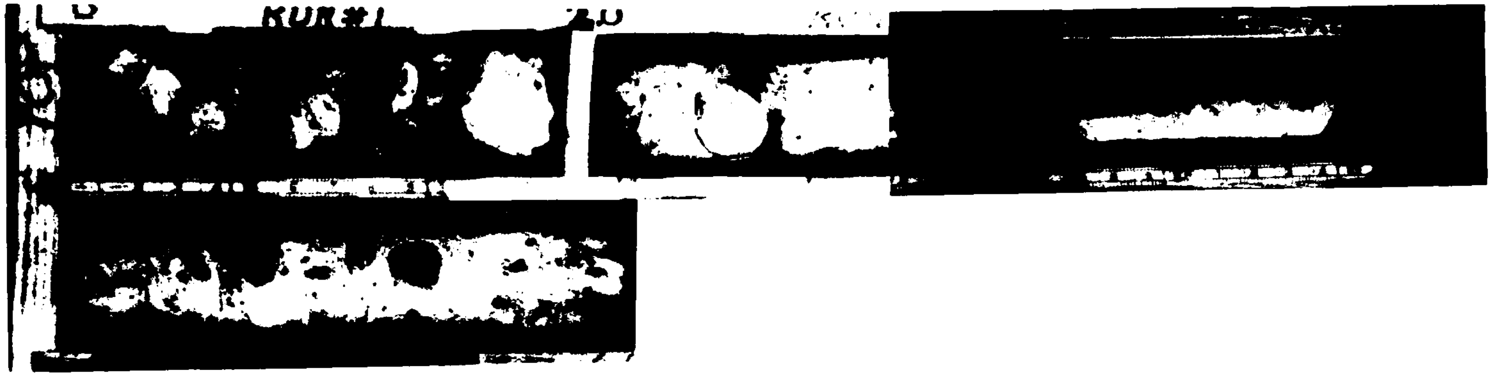
Boring No. R-2-82

Elev 788.0	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.0-0.1'					
	1.0							
				154.4	14705	9080		
	2.0							
	3.0							
	4.0							
	5.0							
	6.0							
				149.4	14202	9530		
	7.0							
	8.0							
			Bottom of boring					

Boring No. R-5-82

Elev 788.0	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.0-2.0' Wht reaction product coating many breaks					
	1.0							
786.0	2.0							
	3.0		Bottom of boring 9.8'					

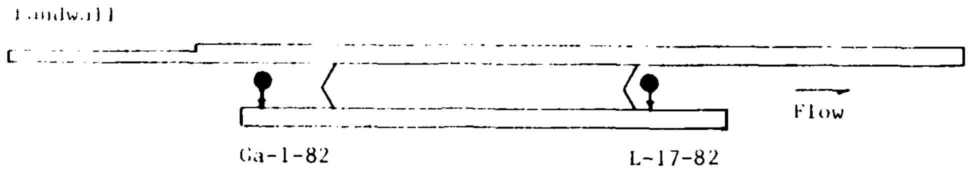
☒ Test specimen location



Concrete core, vertical boring R-5-82 in lower guard wall, 0.0-2.0' damaged zone

Concrete core, vertical boring R-2-82 in upper guard wall, 0.0-0.1' damaged zone

CONCRETE PROPERTIES
Lock #7 Mon River



Horizontal Boring Locations

No Scale

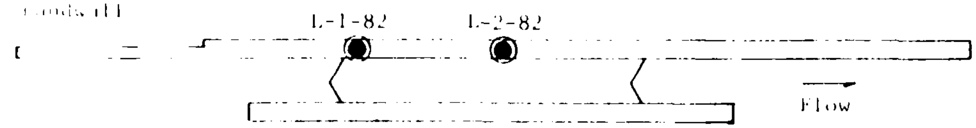
Boring No. Ga-1-82

Elev 783.5	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
778.5			Sound concrete	151.3	14396	10220		
	1.0							
	2.0							
	3.0		Bottom of boring					

Boring No. L-17-82

Elev 768.7	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Sound concrete					
	1.0		Frost damage 0.0-0.2'	149.9	14900	9600		
767.1								
	2.0							
	3.0		Bottom of boring					

CONCRETE PROPERTIES
Lock #7 Mon River



Vertical Boring Locations

No Scale

Boring No. L-1-82

Elev 788.0	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
785.7				151.1	14877	11300		
	5.0							
	10.0		Construction joint					
	15.0							
[illegible]	20.0			150.0	15199	8950		
	25.0							
	30.0							
	35.0							
[illegible]				147.8	14296	6850		
	40.0		Large piece of grout near concrete-rock contact					

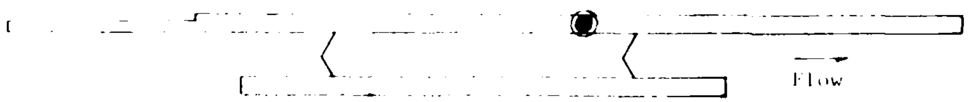
Boring No. L-2-82

Elev 788.0	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.1-1.4'					
			Overlay, 0.0-0.1'					
	5.0		Alkali-silica gel on broken surfaces					
	10.0		Bottom of boring 6.2'					

CONCRETE PROPERTIES
Lock #7 Mon River

Landwall

L-3-82

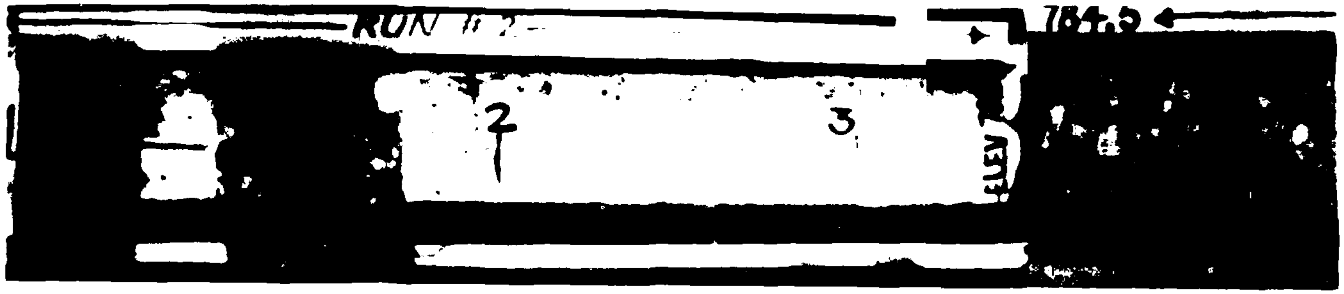


Vertical Boring Locations

			Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	
			Frost damage [illegible]					

Boring No.

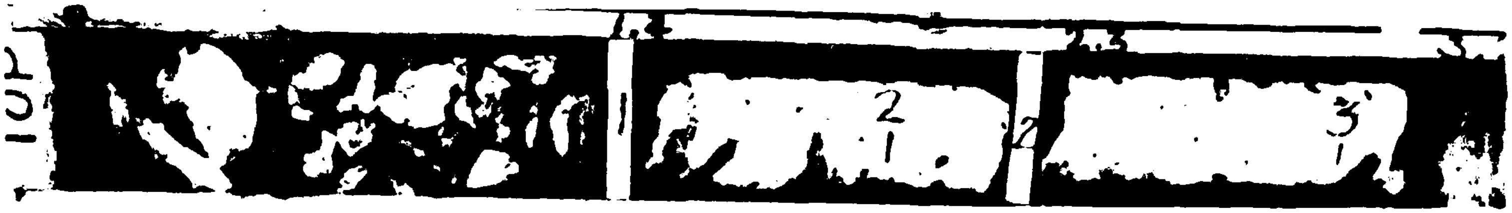
Elev	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν



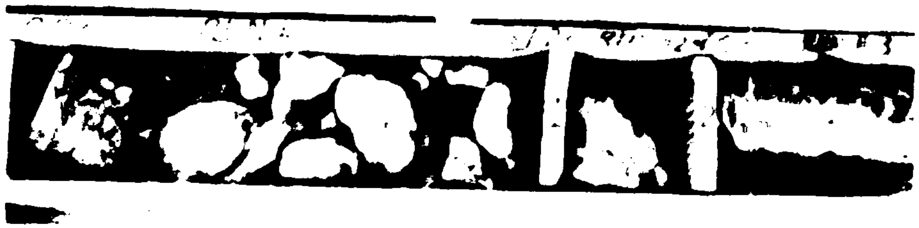
Missing Core

0.0 1.0'

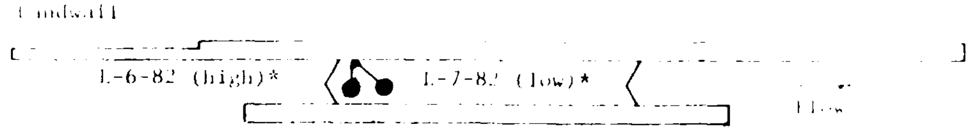
a. Concrete core, vertical boring L-1-82 land lock wall



b. Concrete core, vertical boring [illegible] land [illegible] wall



c. Concrete core, [illegible]



Boring No. PDD L-6-82

Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi
1.5 ft intact concrete non-air-entrained Alkali-silica gel in some voids	149.2	15000	10000	

Boring No. PDD L-7-82

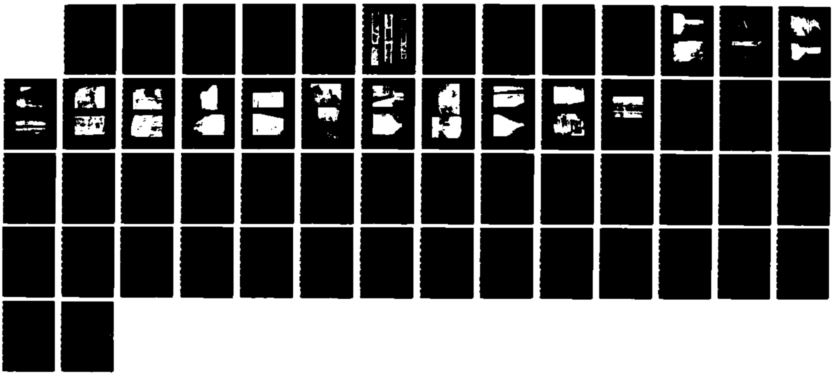
Elev 768.7	Depth	Legend	Material	Unit Wt pcf	Vp fps
			Frost damage 0.0-0.[illegible]	148.8	13681
	1.0				
	2.0				
	3.0		Surface scaled Alkali-silica gel in some voids		

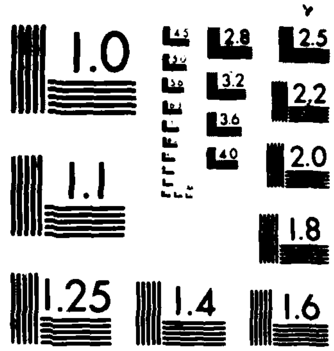
A further column, with a faint header, gives a value of 8050 in the first row.

[illegible] upper pool elevation.
[illegible] lower pool elevation

Plate [illegible]

AD-A182 888 CONDITION SURVEY OF LOCK NUMBER 7 MONONGAHELA RIVER(U) 2/2
ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS
STRUCTURES LAB R L STOWE MAY 87 WES/MP/SL-87-3
UNCLASSIFIED F/G 13/2 NL

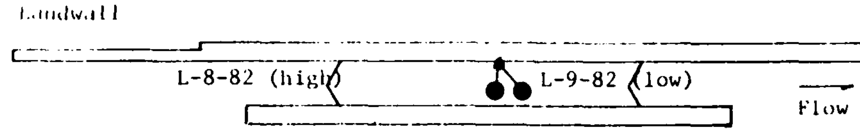




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963-A

CONCRETE PROPERTIES

Lock #7 Mon River



Horizontal Boring Locations

No Scale

Boring No. PDD L-8-82

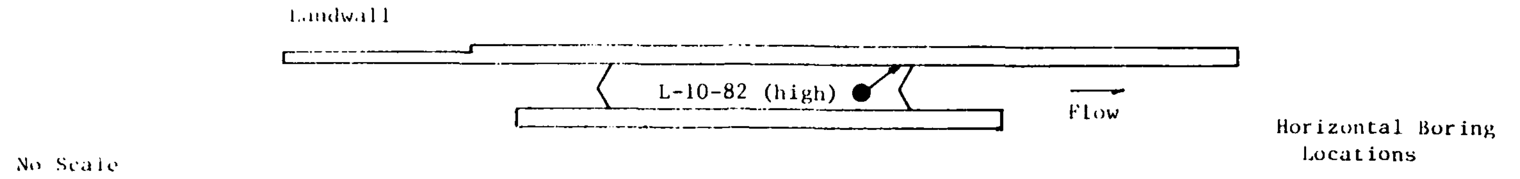
Elev 783.0	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
				148.0	14285	9030		
	1.0							
	2.0		Old concrete Nonair-entrained	149.2	14309	7850		
	3.0		Alkali-silica gel in some voids					
	4.0							

Boring No. PDD L-7-82

Elev 768.7	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
				153.4	14962	10640		
	1.0		Old concrete					
	2.0							
	3.0		3.1 ft intact nonair-entrained Alkali-silica gel in some voids					
	4.0							

CONCRETE PROPERTIES

Lock #7 Mon River



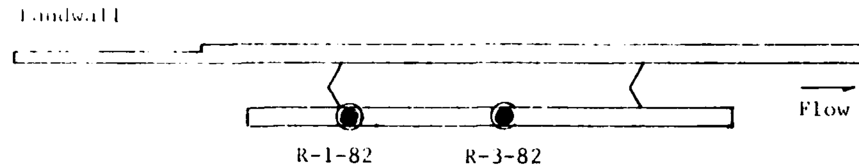
Boring No. L-10-82

Elev 783.0	Depth ft	Leg-end	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Surface scaled	149.4	15214	10250		
	1.0		Nonair-entrained old concrete					
	2.0			148.8	13835	10150		
	3.0							

Boring No.

Elev	Depth	Leg-end	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν

CONCRETE PROPERTIES
Lock #7 Mon River



No Scale

Vertical Boring Locations

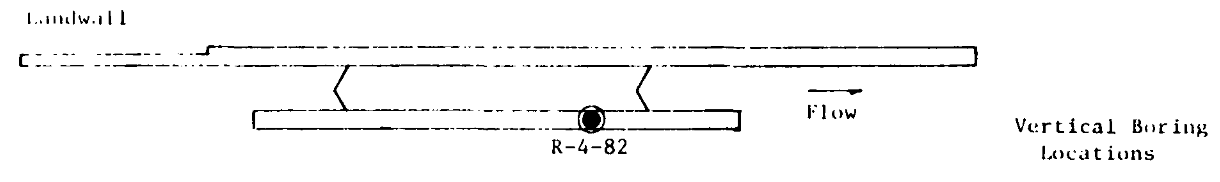
Boring No. R-1-82

Elev 788.0	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.0-1.8' Rubble 0.1-1.0' Wht reaction material on broken surfaces	149.0	14432	7400		
	5.0							
	10.0							
	15.0							
	20.0		Construction joint	149.6	14888	9000		
	25.0							
	30.0		Open crack, unconsolidated Wht reaction material					
	35.0							
	40.0			145.8	14061	5130		

Boring No. R-3-82

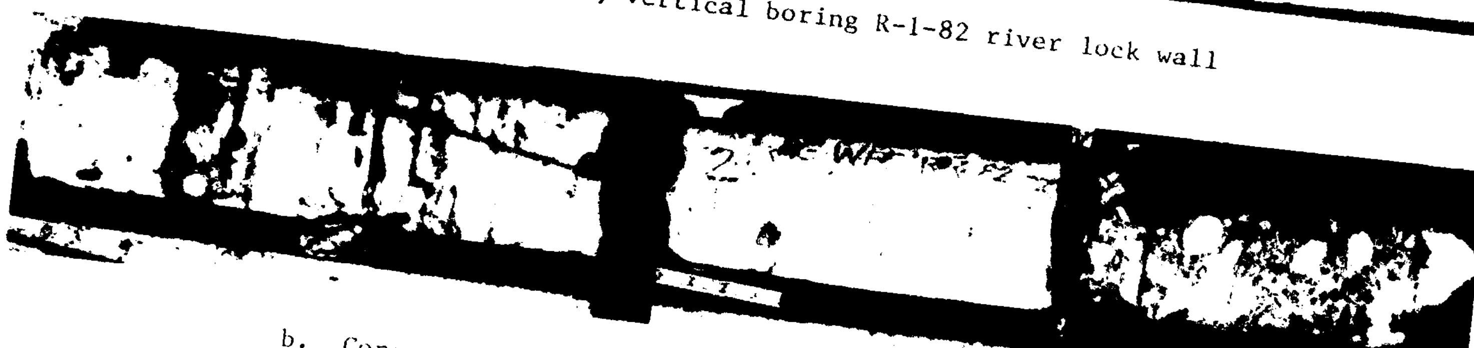
Elev 788.0	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.0-1.3' White reaction product on broken surfaces	150.1	14988	7450		
	5.0							
	10.0							
	15.0							
	20.0			150.3	14493	8300		
	25.0							
	30.0							
	35.0			149.3	14113	6200		
	40.0							

CONCRETE PROPERTIES
Lock #7 Mon River



Boring No. R-4-82									Boring No.								
Elev 788.0	Depth ft	Leg-end	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν	Elev	Depth	Leg-end	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
	5.0 10.0		Frost damage 0.0-0.9' White reaction product on breaks														

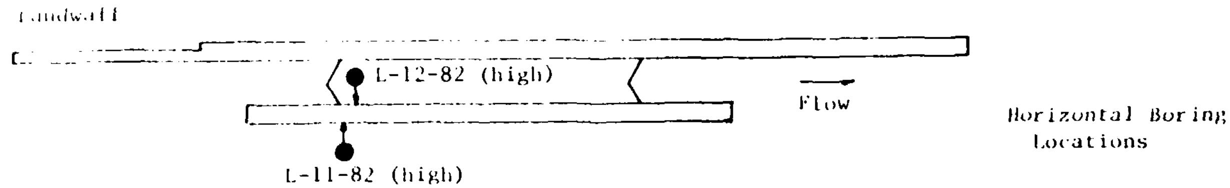
a. Concrete core, vertical boring R-1-82 river lock wall



b. Concrete core, vertical boring R-3-82 river lock wall

c. Concrete core, vertical boring R-4-82 river lock wall

CONCRETE PROPERTIES
Lock #7 Mon River



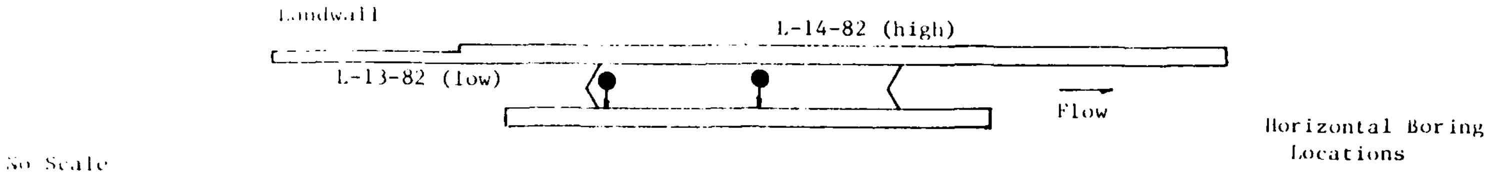
Boring No. L-11-82

Elev 781.0	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Sound concrete	151.8				
	1.0							
	2.0			150.6				
	3.0							

Boring No. L-12-82

Elev 781.0	Depth	Leg-end	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Sound concrete	149.9	14900	9600		
	1.0							
	2.0			150.5	14825	7900		
	3.0							

CONCRETE PROPERTIES
Lock #7 Mon River



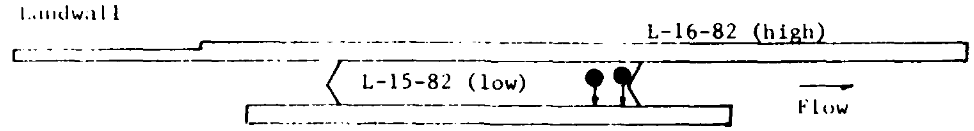
Boring No. L-13-82

Elev	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
768.7			Sound concrete	149.5				
	1.0							
	2.0			148.7	14800	9440		
	3.0							

Boring No. L-14-82

Elev	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
785.0			Frost damage 0.0-0.3'					
	1.0		White reaction product on broken surfaces	147.8	11724			
	2.0			148.5	14669	8690		
	3.0							

CONCRETE PROPERTIES
Lock #7 Mon River



Horizontal Boring Locations

No Scale

Boring No. L-15-82

Elev 768.7	Depth ft	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Sound concrete	146.8	12799	6990		
	1.0		Hairline cracking at 0.2 ft					
	2.0			148.0	14226	7850		
	3.0							

Boring No. L-16-82

Elev 783.0	Depth	Legend	Material	Unit Wt pcf	Vp fps	Comp Strg psi	E x 10^6 psi	ν
			Frost damage 0.0-0.9'					
	1.0		White reaction product on all breaks					
	2.0			142.8	13400	5930		
	3.0							

APPENDIX A

PHOTOGRAPHS OF LOCK NO. 7,
MONONGAHELA RIVER
TAKEN DURING THE
PRELIMINARY STUDY

Photo 1. Top surface of upper guide wall looking downstream.

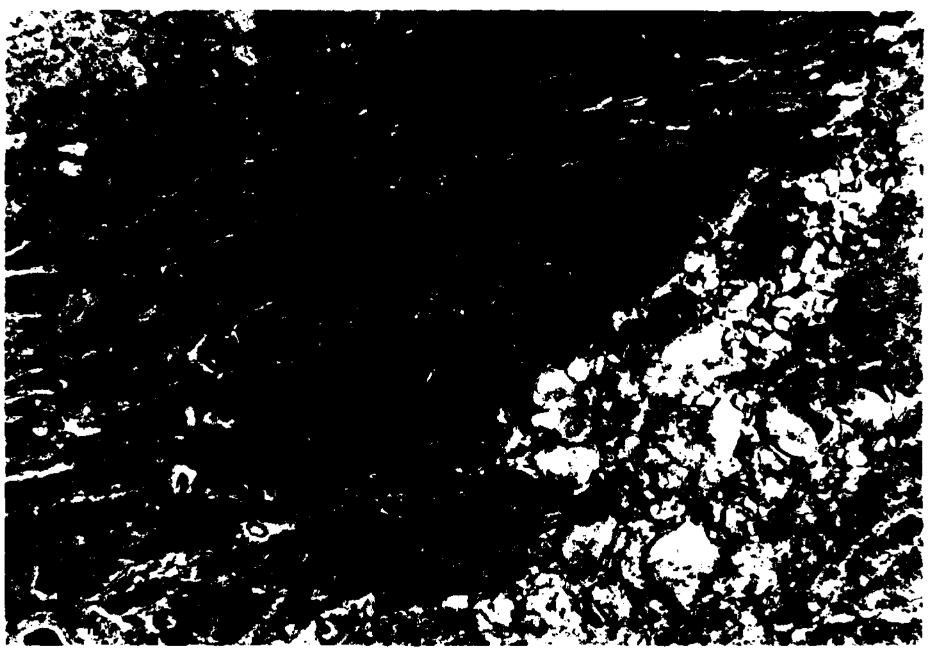


Photo 2. Typical spalling with efflorescence filled cracks and alkali-aggregate reaction rims. Top surface of upper guide wall.

Photo 3. Close-up photo of alkali-aggregate reaction rims [illegible] cence filled random cracks, upper guide wall

Photo 4. Typical vertical surface chamber side of upper guide wall. Large monolith joint spall, horizontal construction joint is spalled, and erosion near waterline

Photo 5. Typical section of backside of the upper guide wall. Architectural exposed aggregate near bottom of picture

Photo 6. Top view of the lower guide wall looking downstream

Photo 7. Vertical face (chamber side) of the lower guide wall

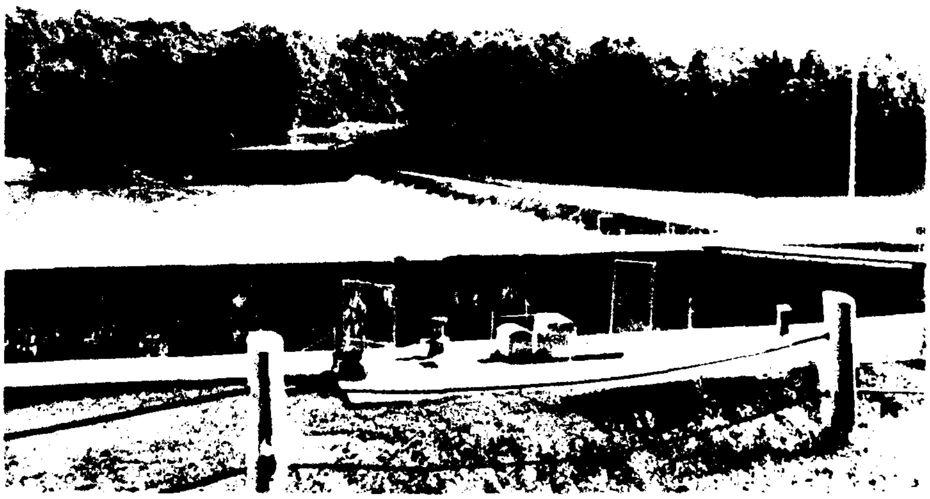


Photo 8. Lower guard wall in midsection of photograph, looking river-ward from left bank

Photo 9. Close-up of lower river wall gate monolith showing severe scaling and extensive spalling

Photo 10. Typical damaged concrete in lower guard wall (chamber side), monoliths R-18 and R-19. (R-19 to right of picture)

Photo 11. Close-up of typical damaged concrete in the lower guard wall (chamber side)

Photo 12. Close-up of typical monolith joint spall and surface damage (chamber side); lower guard wall

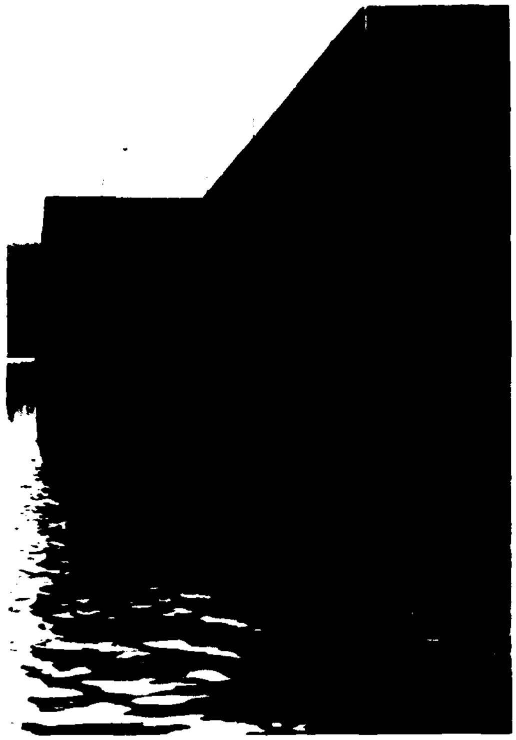


Photo 13. River side lower guard wall looking upstream, dam section in left background

Photo 14. River side lower guard wall (right side of photo) and end portion of the river lock wall. Note the diagonal structural crack in recessed area

Photo 15. River side lower guard wall. Note cracking and leaching in the resurfaced portion of the wall; leaching from older concrete also

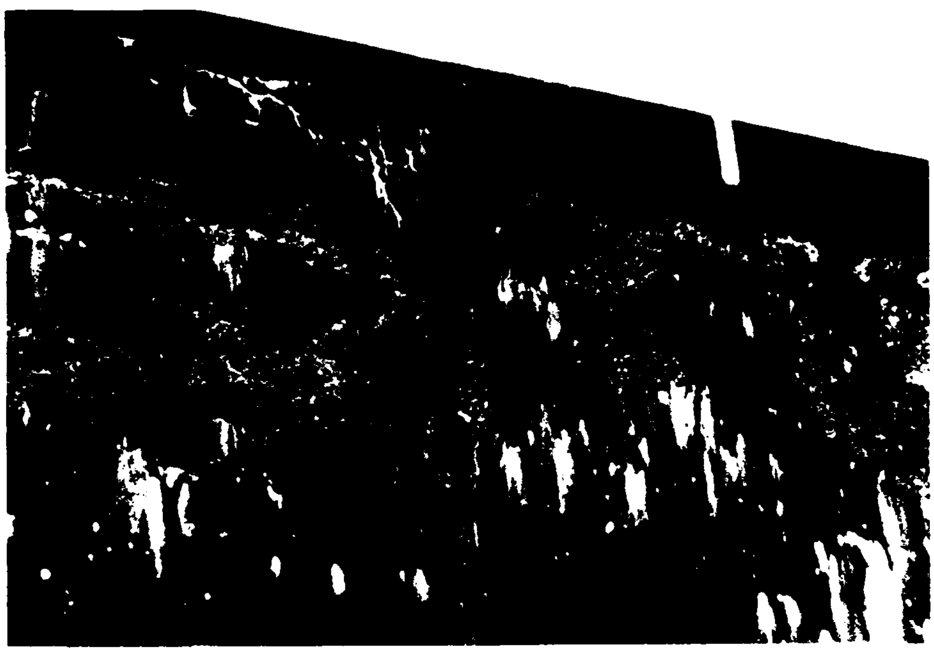


Photo 16. River side lower guard wall showing typical concrete damage (cracking, spalling, leaching, and scaling)

Photo 17. Portion of the land lock wall top surface showing numerous patchings; looking downstream

Photo 18. Portion of the land lock wall top surface showing local spalling and cracking, looking upstream

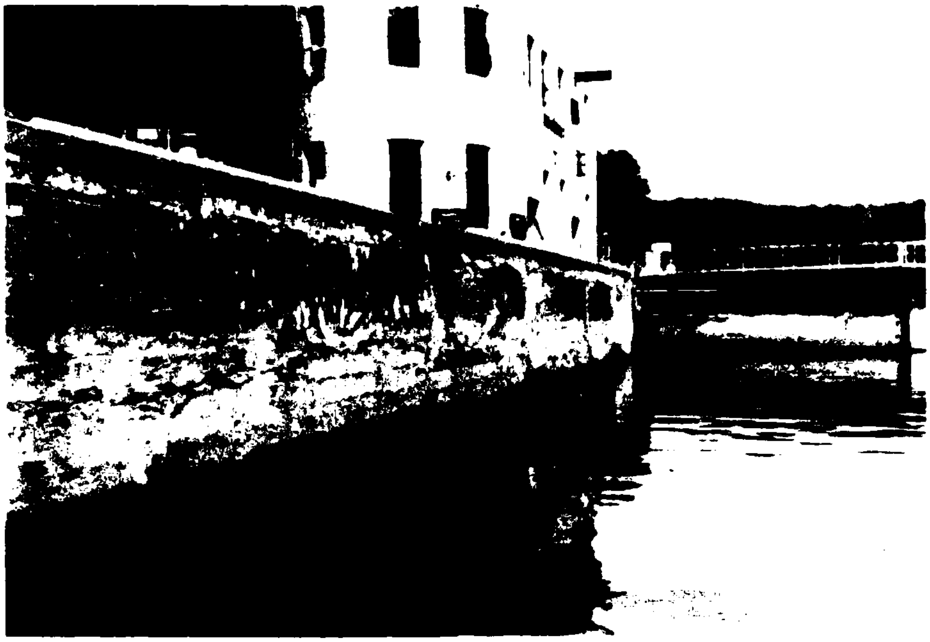


Photo 19. Land lock wall looking downstream with chamber full

Photo 20. Land lock wall looking downstream. Texture appears from scaling (severe to very severe) and monolith joint spalls evident; chamber near emptied

Figure 21. Land lock wall showing drilling operation near upper miter gate. Typical wall concrete deterioration

Photo 22. Close-up of spalling and erosion effects around ladderway and line hook casting, land lock wall looking upstream

Photo 23. River lock wall looking riverward with chamber full

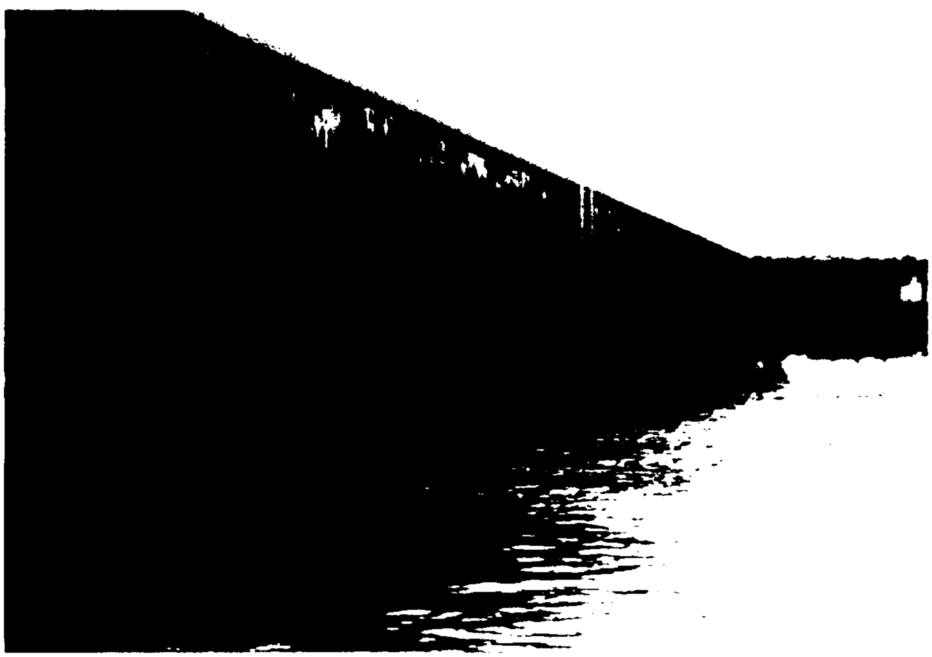


Photo 24. River lock wall looking upstream

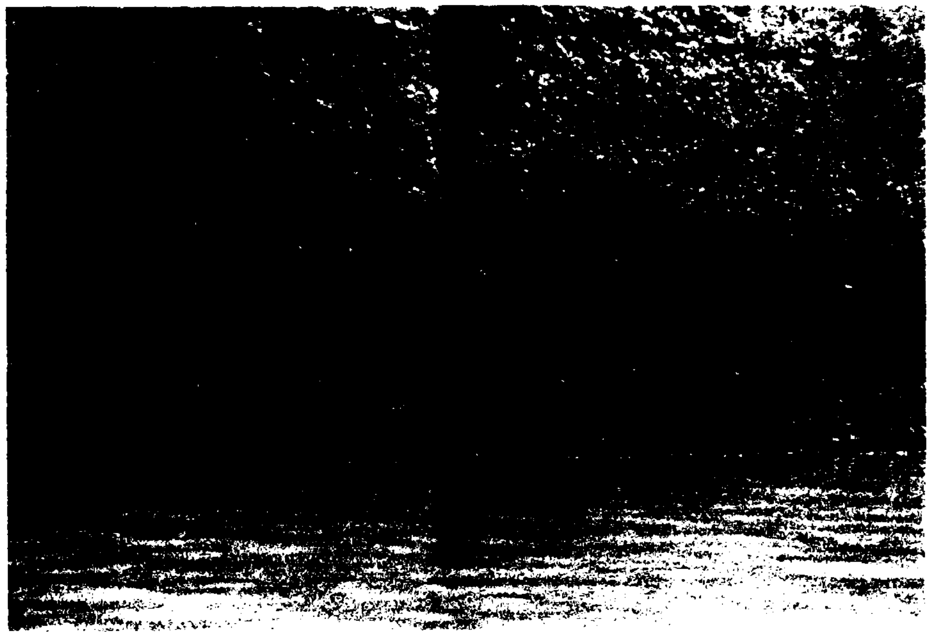


Photo 25. River lock wall showing scaling and monolith joint spalling

Photo 26. River side of river lock wall showing horizontal cracking and efflorescence

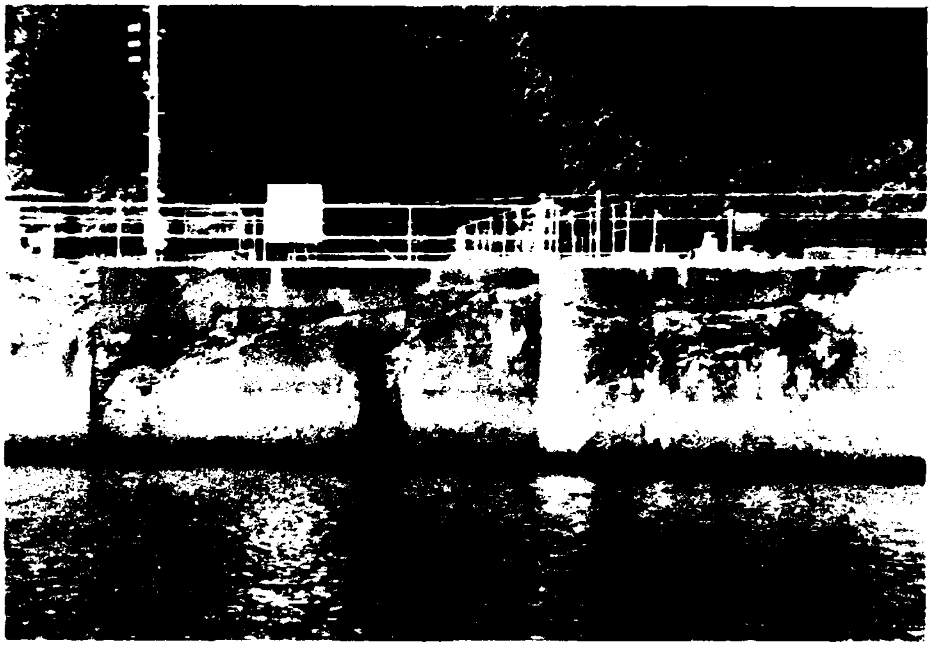


Photo 27. River side river lock wall showing diagonal structural crack, scaling, spalling, and efflorescence

APPENDIX B

FIELD DRILLING LOGS
LOCK NO. 7, MONONGAHELA RIVER

Hole No. NC WES L-1-82

DRILLING LOG	DIVISION Pittsburgh District	INSTALLATION L & D #7 Monongahela River	SHEET 1 OF 2 SHEETS

1. PROJECT: Condition Survey Lock #7
2. LOCATION (Coordinates or Station): 4' U/S & 3' Back Lock Wall Mon.Jt.L-12/L-13
3. DRILLING AGENCY: Mobile
4. HOLE NO (As shown on drawing title and file number): NC WES L-1-82
5. NAME OF DRILLER: Sammy Bowelen
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF CONCRETE: 43.1'
8. DEPTH DRILLED INTO ROCK: 29.7'
9. TOTAL DEPTH OF HOLE: 72.8'
10. SIZE AND TYPE OF BIT: 6 x 7 3/4 (6" CORE)
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
12. MANUFACTURER'S DESIGNATION OF DRILL: Failing 43 SA
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES: 19
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 10-5-82 | COMPLETED 10-19-82
17. ELEVATION TOP OF HOLE: 788.0
18. TOTAL CORE RECOVERY FOR BORING: 100 %
19. SIGNATURE OF INSPECTOR: Alberto Laborda

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
788.0	0.0					Top of Wall
	1.0	Concrete		100		
	3.5			100	1	
	6.0			100		
					2	
	10.5			100		
	11.8			100		
					3	LEGEND
	14.4			100		Concrete
					4	Shale
	18.7			100		Indurated Clay
					5	Break
	23.7			100		
					6	Fractured Zone (rubble)
	26.9			100		Siltstone
	29.3			100		
					7	Sandstone
	34.0			100	8	Limestone
						Calcareous
					9	Nodule
	39.5			100		
					10	
744.9	43.1			100		Bottom of Concrete

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) | PROJECT | HOLE NO

Hole No. NG WES L-1-82

DRILLING LOG	DIVISION Pittsburgh District	INSTALLATION L & D #7 Monongahela River	SHEET 2 OF - SHEETS

1. PROJECT: Condition Survey Lock #7	10. SIZE AND TYPE OF BIT: [illegible] in" CORE
2. LOCATION (Coordinates or Station): 4' U/S & 3' Back Lock Wall Mon.Jt.L-12/L-13	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: Mobile	12. MANUFACTURER'S DESIGNATION OF DRILL: Failing 43 SA
4. HOLE NO. (As shown on drawing title and file number): NG WES L-1-82	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: Sammy Bowelen	14. TOTAL NUMBER CORE BOXES: [illegible]
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED [illegible] COMPLETED [illegible]
7. THICKNESS OF CONCRETE: 43.1'	17. ELEVATION TOP OF HOLE: [illegible]
8. DEPTH DRILLED INTO ROCK: 29.7'	18. TOTAL CORE RECOVERY FOR BORING: [illegible] %
9. TOTAL DEPTH OF HOLE: 72.8'	19. SIGNATURE OF INSPECTOR: Alberto Laberia

ELEVATION 748 a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
744.9	43.1			100		Bottom of Concrete
			SHALE Mod. hard, grey			
742.5	46.3 / 46.5			100	11	
			LIMESTONE Hard, med grey; Indurated clay			
	49.7			100	12	
736.2 / 736.0			INDURATED CLAY mod. hard, med grey			
			SILTSTONE Hard, med. grey; Sandstone		13	
733.1	54.6			100		
			SANDSTONE Hard, grey		14	
	58.8			100	15	
					16	
	63.8			100		
			Indurated clay; Indurated clay		17	
721.9			INDURATED CLAY Mod. hard to soft, med. grey			
	68.2			100	18	
715.1	72.8			100	19	Bottom of Hole

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) | PROJECT | HOLE NO.

Hole No.

DRILLING LOG	DIVISION: Pittsburgh District	INSTALLATION: Lock & Dam #7 Mon	SHEET OF SHEETS

1. PROJECT: New Geneva, Pa Lock & Dam #7	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station): 2' U/S #4-back River wall Mon. Jt R-5, R-4	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
3. DRILLING AGENCY: CE Mobile	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. (As shown on drawing title and file number): NG WESR-1-82	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
5. NAME OF DRILLER: Jerry Trim	14. TOTAL NUMBER CORE BOXES: 19
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 10-22-82 / COMPLETED
7. THICKNESS OF CONCRETE: 41.6	17. ELEVATION TOP OF HOLE: 788.0'
8. DEPTH DRILLED INTO ROCK: 31.3	18. TOTAL CORE RECOVERY FOR BORING: %
9. TOTAL DEPTH OF HOLE: 72.9'	19. SIGNATURE OF INSPECTOR: Alberto Laborda

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
788.0	0.0					Top of Wall
	1.0			100		
	1.8			100		
	3.8			100	1	
	6.3			100		
					2	
	11.1			100	3	
	14.0			100	4	
	17.6			100		
					5	
	22.6			100	6	
	25.0			100	7	
	28.6			100	8	
	33.3			100	9	
	37.3			100	10	
746.4	41.6		Contact loose	100	11	Bottom of Concrete

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE | PROJECT | HOLE NO.

Hole No. NG WES R-1-82

DRILLING LOG	DIVISION Pittsburgh District	INSTALLATION Lock & Dam #7 Monongahela River	SHEET 2 OF 2 SHEETS

1. PROJECT: New Geneva, Pa Lock & Dam #7
2. LOCATION (Coordinates or Station): 2' U/S #4-back River wall Mon. Jt R-5, R-4
3. DRILLING AGENCY: CE Mobile
4. HOLE NO (As shown on drawing title and file number): NG WESR-1-82
5. NAME OF DRILLER: Jerry Trim
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT
7. THICKNESS OF CONCRETE: 41.6'
8. DEPTH DRILLED INTO ROCK: 31.3'
9. TOTAL DEPTH OF HOLE: 72.9'
10. SIZE AND TYPE OF BIT: 5 X 7 3/4
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
12. MANUFACTURER'S DESIGNATION OF DRILL:
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED / UNDISTURBED
14. TOTAL NUMBER CORE BOXES: 19
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 10-22-82 / COMPLETED
17. ELEVATION TOP OF HOLE: 788.0'
18. TOTAL CORE RECOVERY FOR BORING: %
19. SIGNATURE OF INSPECTOR: Alberto Laborda

ELEVATION (a)	DEPTH (b)	LEGEND (c)	CLASSIFICATION OF MATERIALS (Description) (d)	% CORE RECOVERY (e)	BOX OR SAMPLE NO (f)	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) (g)
745						
746.4	41.6					Bottom of Concrete
			SHALE Mod. hard, dk. gray			
	44.7			100	12	
	48.2			100	13	
739.5	48.5		INTERBEDDED SANDSTONE AND SHALE Hard, grey Indurated clay Sandstone Shale			
	51.1			100	14	
733.1	54.9			100	15	
			SANDSTONE Hard, med grey			
	58.9			100	16	
	62.9		soft clay seam	100	17	
720.8	67.2			100	18	
			INDURATED CLAY Mod hard to soft, gray			
	69.3			100	19	
715.0	73.0			100	20	Bottom of Hole

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) | PROJECT | HOLE NO

Hole No. NG WES R-3-82

DRILLING LOG	DIVISION Pittsburg District	INSTALLATION Lock & Dam #7 Monongahela River	SHEET 1 OF 2 SHEETS

1. PROJECT New Geneva, Lock & Dam #7	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) 86' U/S of River Wall gate recess 3' in from lock wall	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. (As shown on drawing title and file number) NG WES R-3-82	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED: — UNDISTURBED:
5. NAME OF DRILLER Sammy Bowden	14. TOTAL NUMBER CORE BOXES 14
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED: — COMPLETED:
7. THICKNESS OF CONCRETE 40.6'	17. ELEVATION TOP OF HOLE 788.0
8. DEPTH DRILLED INTO ROCK 34.4'	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 75.0'	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
788.0	0.0					Top of Wall
	[illegible]			100		
	1.7			100		
	3.0			100		
	5.3			100	1	
	7.6			100	2	
	11.5			100	3	
	16.4			100	4	
	21.0			100	5	
	26.1			100	6	
	30.5			100		
	33.5			100	7	
	38.2			100		
[illegible]73	40.7				8	Bottom of Concrete

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) | PROJECT | HOLE NO

Hole No. NG WES R-3-82

DRILLING LOG	DIVISION Pittsburg District	INSTALLATION Lock & Dam #7 Monongahela River	SHEET 2 OF 2 SHEETS

1. PROJECT New Geneva, Lock & Dam #7	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station) 86' U/S of River Wall gate recess 3' in from lock wall	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL
3. DRILLING AGENCY	12. MANUFACTURER'S DESIGNATION OF DRILL
4. HOLE NO. (As shown on drawing title and file number) NG WES R-3-82	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED
5. NAME OF DRILLER Sammy Bowden	14. TOTAL NUMBER CORE BOXES 14
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED — COMPLETED
7. THICKNESS OF CONCRETE 40.6'	17. ELEVATION TOP OF HOLE 788.0
8. DEPTH DRILLED INTO ROCK 34.4'	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 75.0'	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
7565						
747.3	40.7					Bottom of Concrete
	41.5		SHALE Mod. hard, med. grey	100		
743.0	45.0					
	46.0		INTERBEDDED SILTSTONE AND SHALE Hard, grey	100	9	
737.4	51.6		SANDSTONE Hard, lt. grey	100	10	
	55.4			100		
					11	
	59.7			100		
725.0	63.0			100	12	
	63.5		INDURATED CLAY Mod. hard to soft, lt. grey			
	68.7			100		
					13	
	71.8			100		
715.0	73.0		SANDSTONE			
714.0	74.0		INTERBEDDED SANDSTONE AND SHALE			
713.0	75.0			100	14	Bottom of Hole

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE — PROJECT — HOLE NO

APPENDIX C

ROCK PETROGRAPHIC REPORT
LOCK NO. 7, MONONGAHELA RIVER

Samples

1. Three borings were made in the foundation rock. Ninety-five feet of rock core were extracted from them for examination and testing. The physical tests of the rock are described elsewhere and will not be discussed here. This report will cover the petrographic examination of the rock that was made.

2. Two cores were from the river wall and one was from the land wall. Structures Laboratory (SL) serial numbers were assigned to the cores as described below:

SL Serial No.	Field Serial No.	Sample Identification
PITT-10 DC-25-33	NG-WES-L-1-82	Rock, 43.1-ft to 72.4-ft depth. Vertical hole from top of land wall.
PITT-10 DC-35-42	NG-WES-R-1-82	Rock, 41.6-ft to 73.0-ft depth. Vertical hole from top of river wall.
PITT-10 DC-43-48	NG-WES-R-3-82	Rock, 40.6-ft to 75-ft depth. Vertical hole from top of river wall.

Test procedure

3. The cores were examined and logged with emphasis on homogeneity and competence of material. If the cores were sealed in wax, a slit was made in the protective wax coating to permit visual examination. This also permitted simple chemical and physical tests to be made on the exposed core surface. A dissecting needle was used to probe the specimen to determine relative hardness and consistency of material.

4. Dilute hydrochloric acid (HCl) was used to determine the presence of carbonate by applying a drop or drops of it along the lengths of rock core at selected locations. If it fizzed, there was carbonate present.

5. Samples representing the different rock types from different depths were air dried and put into water to promote disaggregation. Portions of some samples having like characteristics were examined in more detail using a stereomicroscope and X-ray diffraction (XRD).

6. All X-ray diffraction patterns were made with a diffractometer using nickel-filtered copper radiation.

Results

7. The foundation rock was composed of shale, interbedded shale, siltstone and sandstone, sandstone, indurated clay, and then an interbedded shale, siltstone, and sandstone sequence. The lithologic identification of the foundation material is described in field logs L-1-82, R-1-82, and R-3-82 presented in Appendix B.

8. The foundation material 3 to 6 ft directly below the concrete structure was, in general, similar in all three cores. The rock was an olive black (5Y 2/1),(1) fine-grained (particles <0.05 mm) sedimentary rock. This rock tended to slake upon drying but remained intact when immersed in water after drying. The rock was thin bedded with some alternation of silt-size materials and clay-size materials. Some clay-size and silt-size materials were intermixed. The rock will be referred to as shale in this report.

9. The next approximately 6 ft of material consists of a coarser material. More of the material was sand-size particles than present in the above section. The material is transitional due to its interbedded nature. Shale, siltstone, and sandstone are common materials in all three cores. Approximately 5 ft of a section of core from about 46.5-ft to about 51.5-ft depth in Hole L-1 was identified as limestone. This limestone rock was competent. Its composition differed from the material in other holes at this depth by containing more calcite. This section was limestone or a calcareous shale or siltstone or sandstone, but will be referred to as limestone for convenience.

10. The next 10 ft of material was a competent sandstone. A couple of thin seams interrupted what otherwise was a continuous bed of sandstone. Thin clay seams were identified at 62.5-ft depth in Hole R-1 and at 62.7-ft depth in Hole R-3.

11. Below the sandstone there is a sequence of moderately hard, indurated clay. The rock is similar in all three holes. The material disaggregates easily in water and was generally fractured when it was removed from a core barrel.

12. Hole R-3 was the only one with material recovered from below the indurated clay. Some sandstone and interbedded sandstone and shale were identified in this deeper interval. Quartz was common to all the samples examined by XRD and clay minerals were present in all samples except core R-3 at 62.2 ft, which represented sandstone.

13. Table C-1 shows the distribution of constituents in the samples examined. Some of the mineralogical identifications were only tentative, as they were based on the presence of a single peak. The information did aid in the correlation of lithology between holes as the peaks were unique and common to a particular rock type as in samples L-1 at 62.4 ft and R-3 at 63.6 ft; both contain a mixed-layer clay.

(1) The Rock-Color Chart Committee, E. M. Goddard, Chm., "Rock-Color Chart," 1975, The Geological Society of America, Boulder, CO.

Conclusions

14. Each of the three rock cores consisted of four lithologies. In increasing depth, these were shale, a calcareous transitional zone, sandstone, and indurated clay. Elevations of these zones were similar between the cores.

Table C-1

Mineralogical Composition of Selected Rock Core Samples from Lock and Dam No. 7

Constituents	Samples						
	R-1 45.5 ft, Shale	L-1 48.0 ft, Limestone	R-1 51.1 ft, Inter-bedded Material	R-3 46.5 ft, Inter-bedded Material	R-3 62.2 ft Sandstone	L-1 62.9 ft, Indurated Clay	R-3 63.6 ft, Indurated Clay
Nonclays							
Quartz	X	X	X	X	X	X	X
Calcite	X	X	--	--	X	X	X
Plagioclase feldspar	X	X	X	X	X	X	--
Siderite	X†	X†	--	X†	--	--	--
Marcasite	X†	--	--	--	--	X†	--
Pyrite	--	--	--	--	X	--	--
Clays							
Clay-mica	X	X	X	X	--	X	X
Chlorite	X*	X*	--	X	--	X	X*
Kaolinite	X**	X**	X**	X**	--	X**	X**
Palygorskite	--	X°	X°	--	--	--	--
Mixed-layer	--	--	--	--	--	X°°	X°°

† Based on identification of one line.
* 14-A clay may also be vermiculite or smectite.
** Not positively identified due to possible presence of chlorite.
° Based on 10.5-A peak.
°° Based on broad peak at 10-11 A.

APPENDIX D

CONCRETE PETROGRAPHIC REPORT
LOCK NO. 7, MONONGAHELA RIVER

Samples

1. Portions of concrete core from two vertical and seven horizontal cores from Lock and Dam No. 7 were selected for detailed petrographic examination. The cores were 6 in. in diameter. The cores identified below are samples examined in detail. Other cores that were logged are described elsewhere.

Structures Laboratory (SL) Serial No.	Field Identification No.	Description
PITT-10 CON-75	L-5-82	Vertical core on land lock wall, 200 ft upstream of end of guide wall and 4-1/2 ft from vertical face. Sample taken from 0.0- to 3.0-ft depth.
PITT-10 CON-76-86	R-1-82	Vertical core on river lock wall 2 ft upstream of monolith joint R-4/R-5 and 4 ft from vertical face. Samples taken from 0.0- to 0.1-, 1.4- to 1.6-, 14.0- to 14.4-, 20.5- to 21.4- and 40.0- to 41.6-ft depths.
PITT-10 CON-108	PDD L-6-82	Horizontal core in land wall upstream gate recess, 1 ft downstream of L-12/L-13 monolith joint, 7 ft down. Samples taken from 0.0- to 0.05- and 1.06- to 1.5-ft depths.
PITT-10 CON-107	PDD L-7-82	Horizontal core in land wall upstream gate recess, 1 ft upstream of recess and 4.5 ft above water level. Samples taken from 0.0- to 0.2- and 1.15- to 2.7-ft depths.
PITT-10 CON-117	PDD L-8-82	Horizontal core on land lock wall, 20 ft downstream of L-16/L-17 monolith joint and 5 ft down. Samples taken from 1.05- to 1.95- and 2.0- to 3.1-ft depths.

Structures Laboratory (SL) Serial No.	Field Identification No.	Description
PITT-10 CON-114	PDD L-9-82	Horizontal core on land lock wall, 5 ft downstream of L-17/L-18 monolith joint and 4.5 ft above water level. Sample taken from 0.0- to 3.1-ft depth.
PITT-10 CON-116	PDD L-14-82	Horizontal core in river lock wall, 20 ft downstream of R-10/R-11 monolith joint and 3 ft down. Samples taken from 0.0- to 0.25-, 1.15- to 1.9-, and 2.9- to 3.0-ft depths.
PITT-10 CON-112	PDD L-16-82	Horizontal core in downstream gate recess on river wall, 5 ft down and one-half the distance under gate machinery. Samples taken from 0.0- to 3.0-ft depth.
PITT-10 CON-115	PDD Gi-3-82	Horizontal core on lower guide wall, 10 ft upstream of L-30/L-31 monolith joint and 4.5 ft above water level. Sample taken from 0.0- to 3.2-ft depth.

Test procedure

2. All of the material shipped, 310.6 ft of concrete and 95.4 ft of rock for a total of 10 vertical and 16 horizontal cores, from Lock and Dam No. 7 was examined and logged in the laboratory. Concrete specimens for petrographic examination were taken from the upper, middle, and lower portions, where available, of nine cores to represent all of the concrete. Representative pieces of cores that contained visual evidence of poorer quality concrete or significant reaction products, as well as pieces of typical concrete, were also selected for detailed examination. The rock is described in a separate report.

3. Freshly broken surfaces, as well as pre-existing fracture surfaces, were examined megascopically and with a stereomicroscope.

4. Two pieces of core were sawed horizontally. One of each pair of sawed surfaces was ground smooth and examined with a stereomicroscope.

5. Cement paste concentrates from typical concrete samples were concentrated by gentle crushing of the concrete and sieving the material over a 45-μm (No. 325) sieve. The material that passed this sieve was backpacked to minimize preferred orientation and examined by X-ray diffraction (XRD). All X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

6. Samples of white reaction products found in some voids and coating some fracture surfaces in the concrete were examined using a stereomicroscope and as immersion mounts using a polarizing microscope.

Results

7. The majority of the concrete was nonair entrained. Only one piece (L-5-82) had a concrete overlay that contained entrained air.

8. The concrete was composed of 2-1/2-in. maximum size natural gravel and sand aggregates. The bulk of the gravel was composed of chert and sandstone with some miscellaneous particles, largely limestone and coal. This composition is typical of gravels and sands from this region.

9. Four cement paste concentrates were examined by XRD. The concentrates were from cores R-1-82, L-8-82, L-14-82, and L-16-82. They were composed of ettringite, hydrogarnet, calcium hydroxide, calcium silicate hydrate, and tetracalcium aluminate carbonate-11-hydrate. These are normal hydrated portland cement phases for such concrete. Calcite, quartz, and some clay were detected. These minerals were likely contamination from aggregate particles in the concrete. No mineralogical components or cement phases were thought to be excessive or deleterious to the concrete.

10. The general condition of the concrete from each core is described in the following paragraphs.

11. Land wall (five vertical cores). Core L-5-82 consisted of a 0.9-ft overlay of newer air-entrained concrete. This overlay consisted of 1-in. maximum size natural gravel composed of well rounded to subangular calcareous rock particles. The fine aggregate was like that found in the other concrete in the structure. A piece of tar paper was found at 0.4-ft depth. This was probably debris inadvertently encapsulated during original construction. The remainder of the concrete in core L-5 was nonair entrained. Directly below the contact of new and old concrete were some old incipient fractures. The fractures were at 1.1- to 1.15-ft depth. This cracking was believed to have been caused by frost action.

12. Four other vertical cores drilled in the land wall also sustained frost damage. Two cores, L-1 and L-2, contained new concrete overlays of 0.5-ft and 0.05-ft thickness, respectively. Two cores, L-3 and L-4, did not contain an overlay. The maximum depth of frost damage was sustained by core L-4 with 1.6 ft of damaged concrete. The other cores sustained damage to a depth of at least 1 ft.

13. Alkali-silica gel was present in both the concrete overlay as well as in the original concrete. The gel was found in air voids and coating old fracture surfaces.

14. River wall (five vertical cores). A 0.05-ft-thick layer of concrete consisting of smaller size aggregate was present on core R-1-82. Since this was not air-entrained concrete, it was believed to be part of the original concrete. The concrete was light olive gray (5 Y 6/1)[(1)] throughout

(1) The Rock-Color Chart Committee, E. M. Goddard, Chm., "Rock-Color Chart," 1975, The Geological Society of America, Boulder, CO.

the length of the core. The concrete in general was well consolidated with no significant segregation. There was some local aggregate segregation and honeycombing at 1.3- to 1.8-ft and at 29.5-ft depths.

15. Alkali-silica gel was found lining voids and fracture surfaces. An abundance of gel was present in the frost-damaged concrete. The amount of reaction product decreased at lower depths but was found throughout the core.

16. Frost damage ranged from a depth of 0.1 ft in core R-2-82 to 2.0 ft in core R-5-82 and was always present.

17. Land wall (eight horizontal cores). Cores from the upper guide wall, lock chamber, and lower guide wall representing the land wall side of the structure were generally all similar in composition. None of the concrete was air entrained. The concrete cores were intact except for PDD L-7-82 and PDD Gi-3-82. Some surface scaling and frost damage to 1 0.15-ft depth was observed in core PDD L-7-82. Core PDD Gi-3-82 was cored through a construction joint and illustrated many construction deficiencies. These included poor consolidation causing honeycombing, noncemented sand filling voids, embedded wood, and numerous pieces of coal in the aggregate. Separation of concrete along the construction joint was common. This resulted in mechanical breaks caused by the drilling.

18. Alkali-silica reaction gel was present in all cores as fillings or linings of voids.

19. The concrete remaining on the vertical surface of the land wall is in reasonable condition as judged by these cores. Some near-surface scaling was present in one core. The deficiencies observed in core PDD Gi-3-82 were not believed to be widespread or significant.

20. River wall (eight horizontal cores). The concrete from the guard wall, lock chamber, and gate recesses represented vertical surfaces of the river wall. Cores PDD L-14-82 and PDD L-16-82 were typical of the concrete from this wall. It was well consolidated, nonair-entrained concrete composed of 1-1/2-in. maximum size coarse aggregate. The paste was light olive gray (5 Y 6/1).[1] The near-surface concrete was fractured and delaminated to a depth of 0.3 ft in core PDD L-14-82 and to a depth of 0.9 ft in core PDD L-16-82. Core L-17-82 showed frost damage to 0.2-ft depth. The other five cores were generally intact. The concrete was in good condition beneath the damaged areas. Breaks in the cores beneath the deteriorated concrete were caused by the coring operations.

21. Alkali-silica reaction gel was found in voids and as coatings along fractured concrete surfaces.

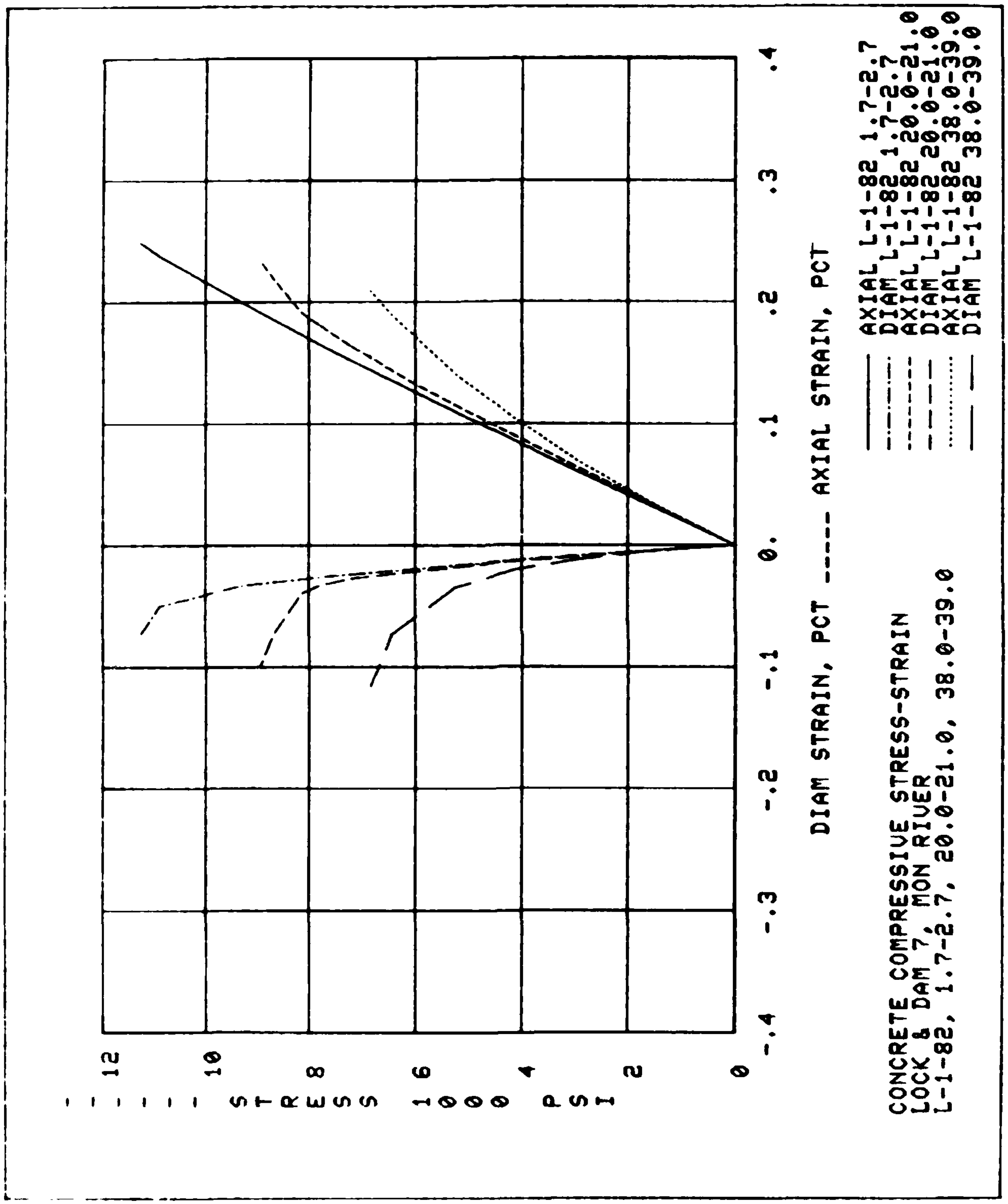
22. The two cores examined represented the most severely damaged concrete taken from the vertical face of the river wall. The concrete from the other river wall cores exhibited some surface spalling and scaling.

Conclusions

23. Logging of 26 concrete cores and detailed examination of portions of 9 of these showed that most showed evidence of frost damage and all showed evidence of alkali-silica reaction. The frost damage was deeper and more widespread on tops of walls; the damage was usually down to a depth of about 1 ft and sometimes to 2 ft. Lock wall damage was more scattered and shallower; depths were usually less than 1 ft. It is possible that damage was due to both frost action and alkali-silica reaction.

APPENDIX E

STRESS-STRAIN GRAPHS FROM
SELECTED CONCRETE CORES,
LOCK NO. 7, MONONGAHELA RIVER



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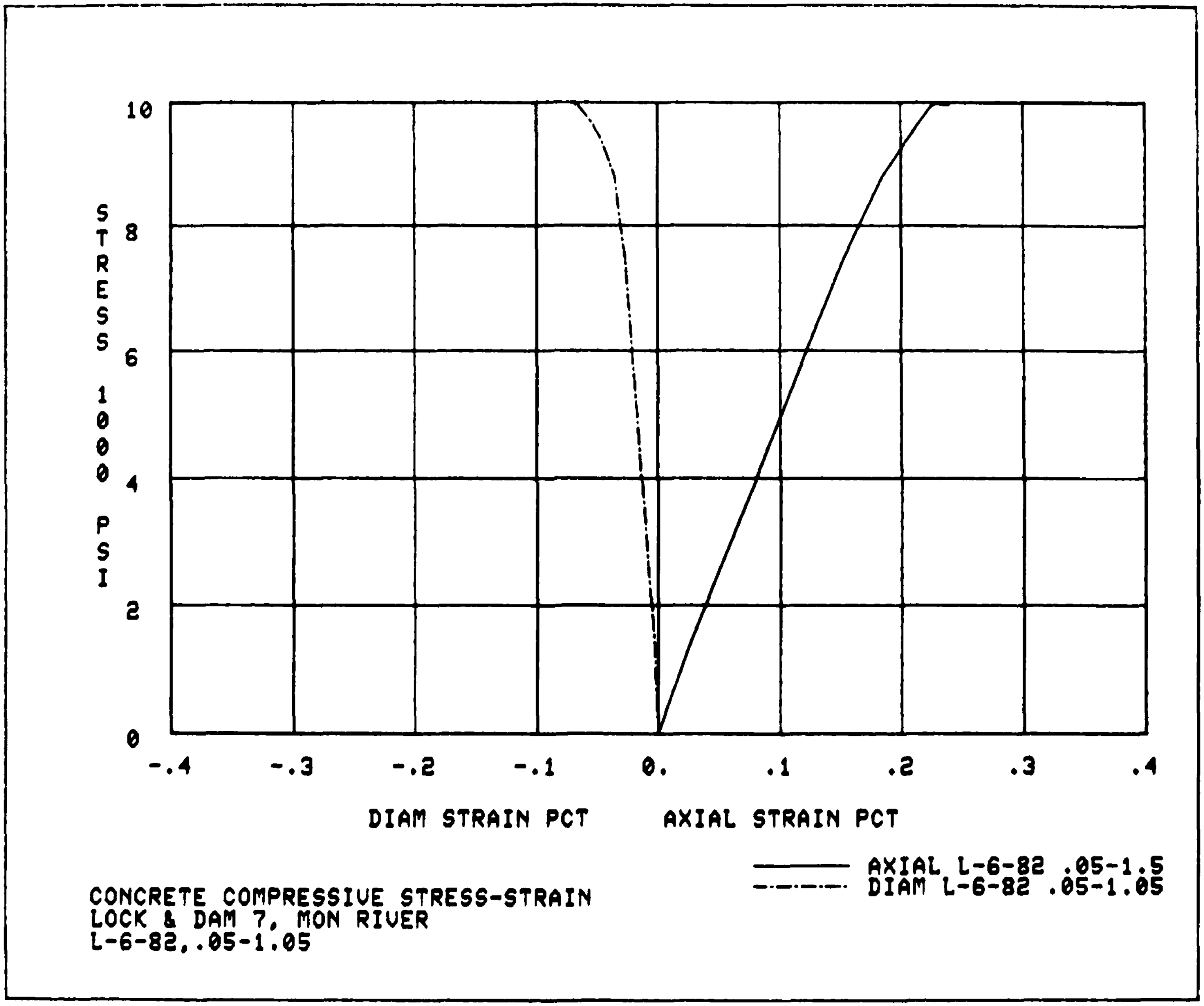
0
2
4
6
8
10
12
-.4
-.3
-.2
-.1
0.
.1
.2
.3
.4
DIAM STRAIN, PCT ----- AXIAL STRAIN, PCT
AXIAL L-1-82 1.7-2.7
DIAM L-1-82 1.7-2.7
AXIAL L-1-82 20.0-21.0
DIAM L-1-82 20.0-21.0
AXIAL L-1-82 38.0-39.0
DIAM L-1-82 38.0-39.0
CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
L-1-82, 1.7-2.7, 20.0-21.0, 38.0-39.0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AXIAL STRAIN PCT
AXIAL L-6-82 .05-1.5
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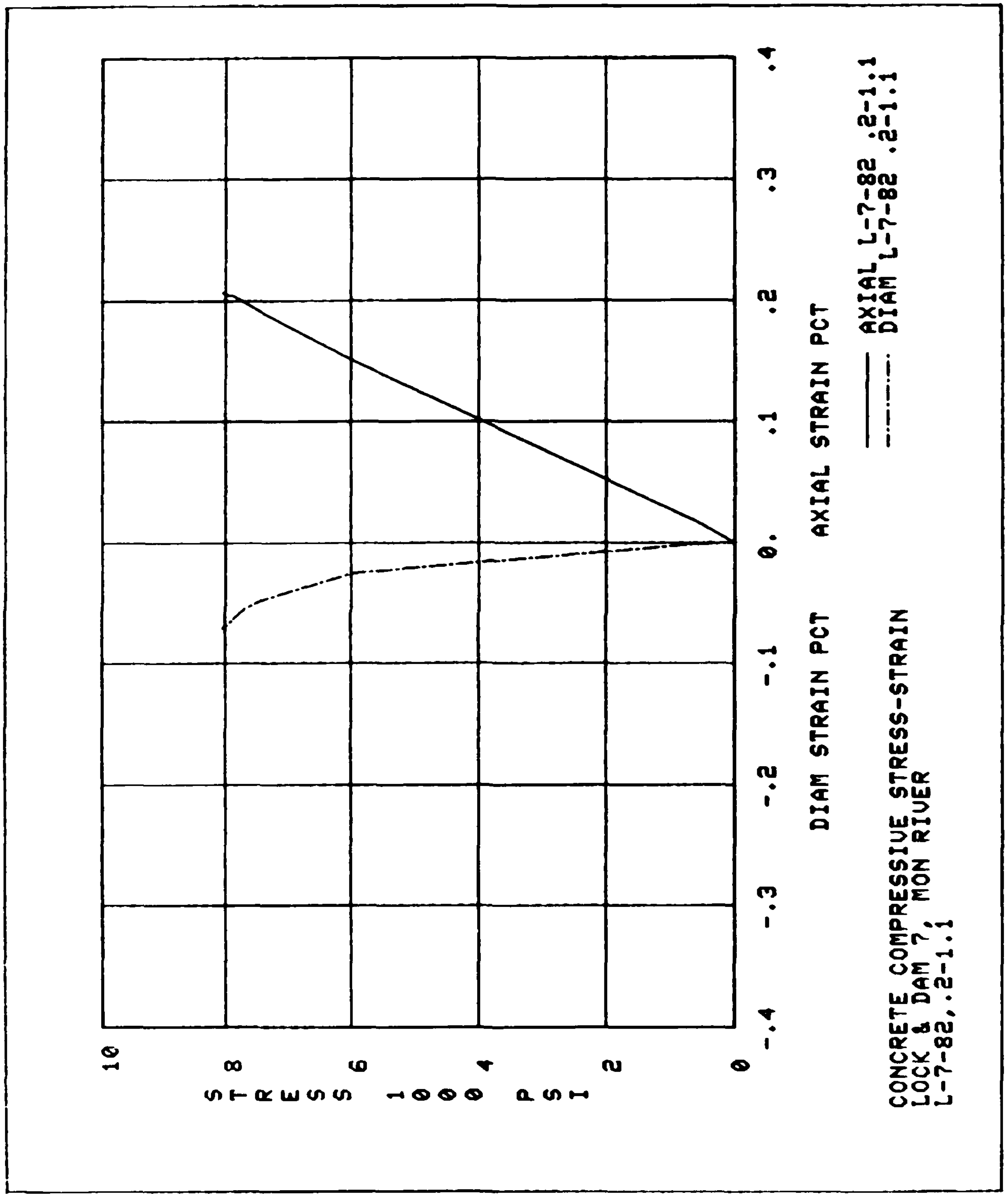
CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
L-6-82,.05-1.05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AXIAL STRAIN PCT
AXIAL L-7-82 .2-1.1
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LOCK & DAM 7, MON RIVER
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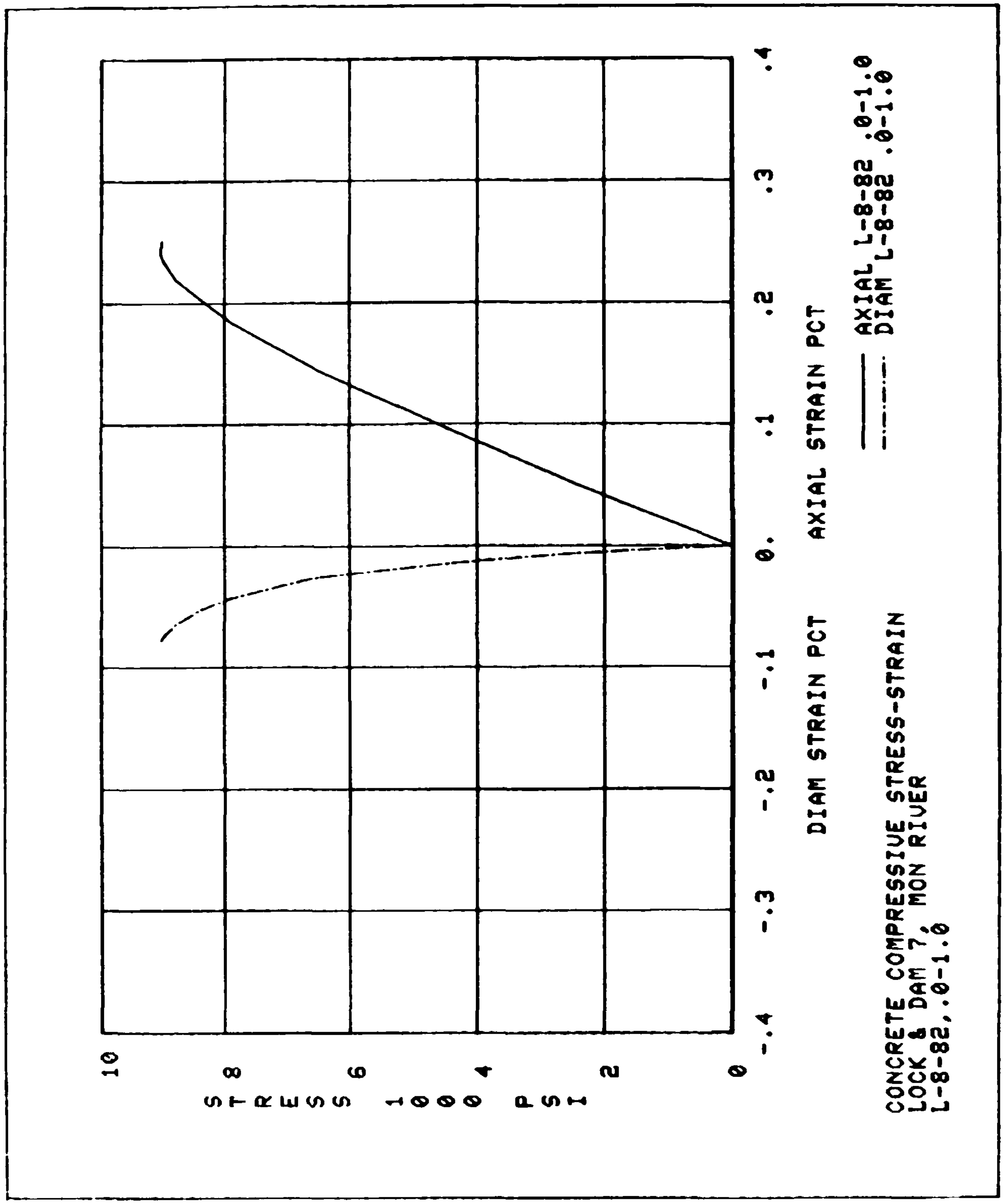
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DIAM STRAIN PCT
AXIAL STRAIN PCT
AXIAL L-8-82 .0-1.0
DIAM L-8-82 .0-1.0
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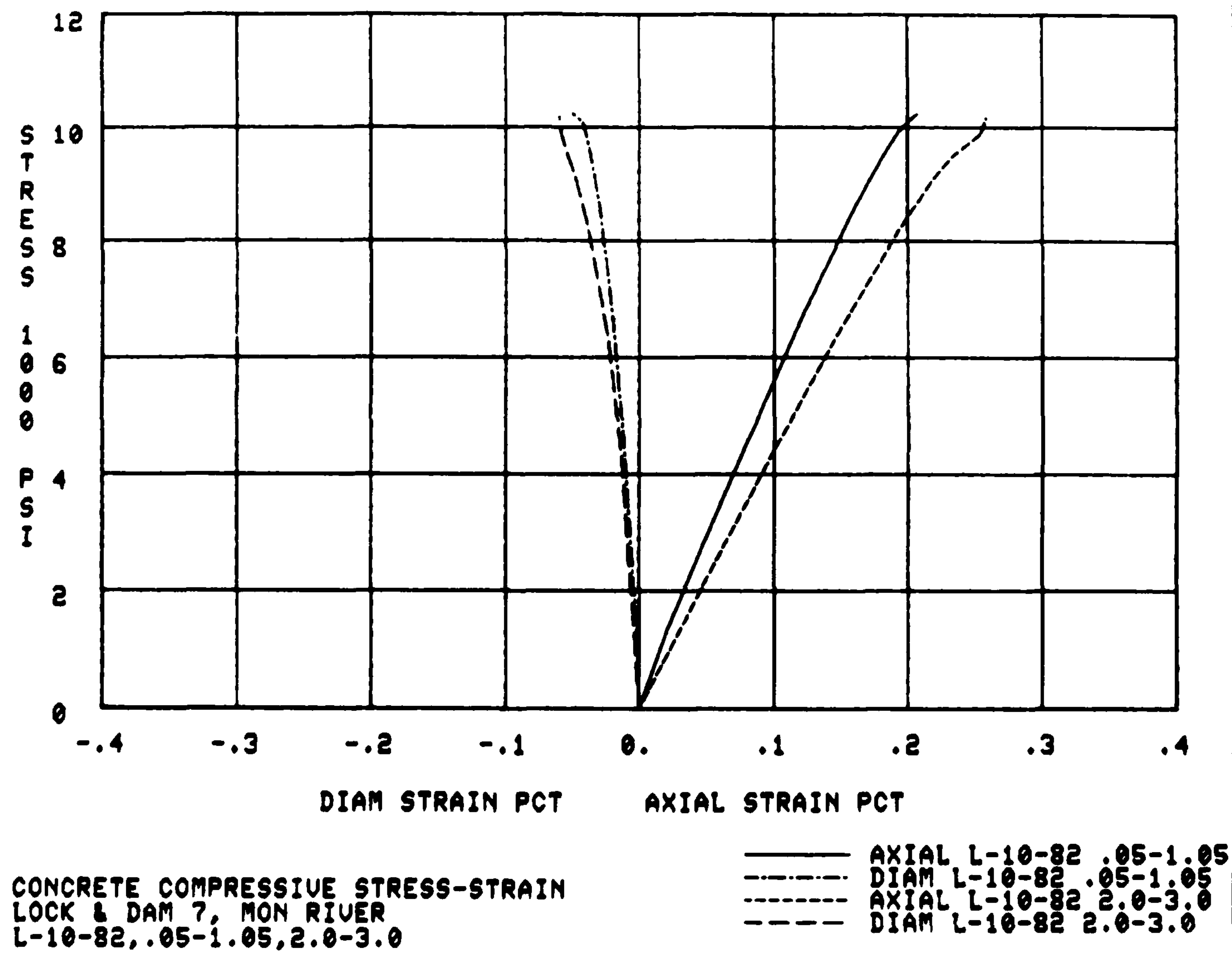
LOCK & DAM 7, MON RIVER
L-8-82,.0-1.0



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.1
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DIAM L-10-82 .05-1.05
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LOCK & DAM 7, MON RIVER
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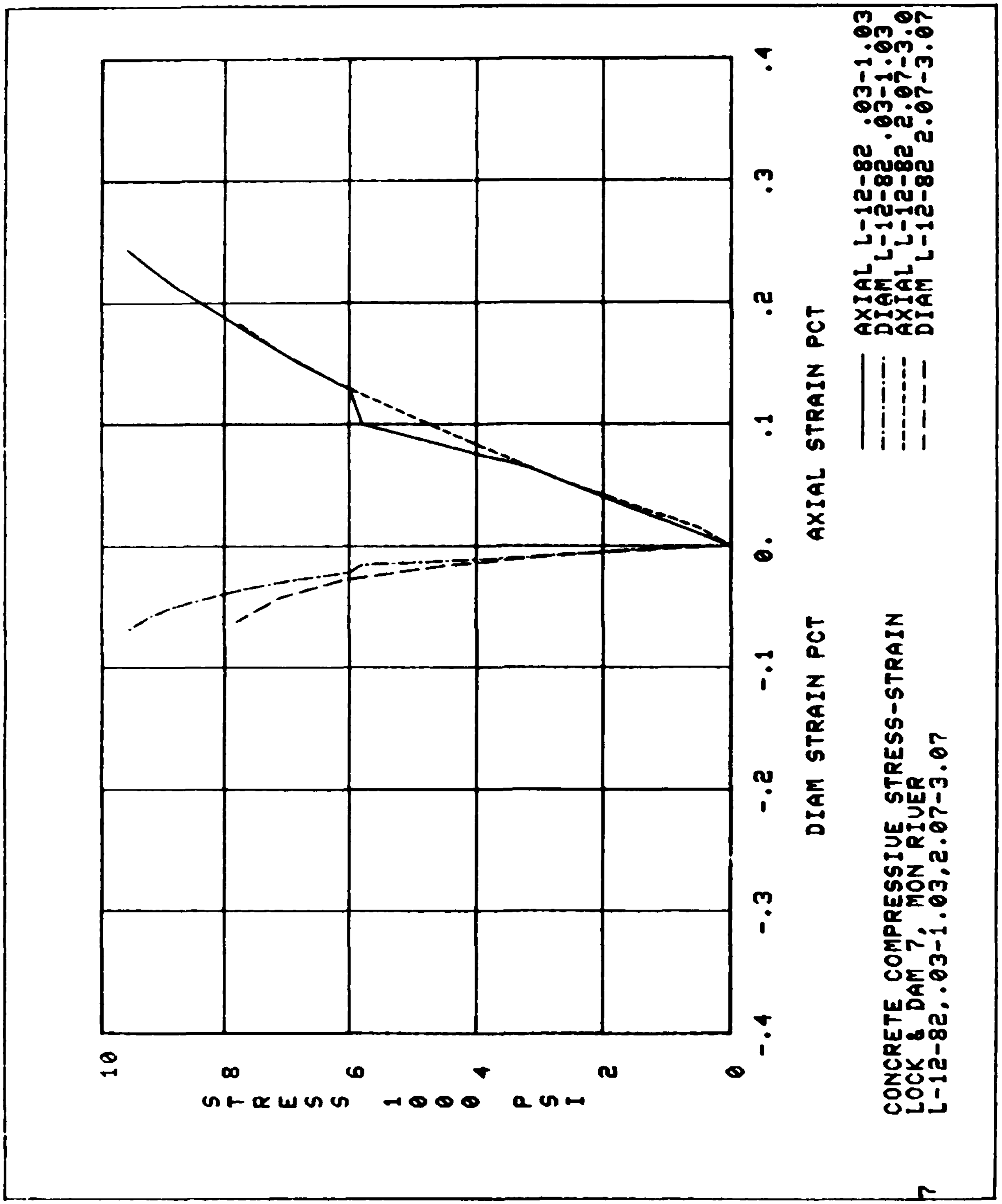
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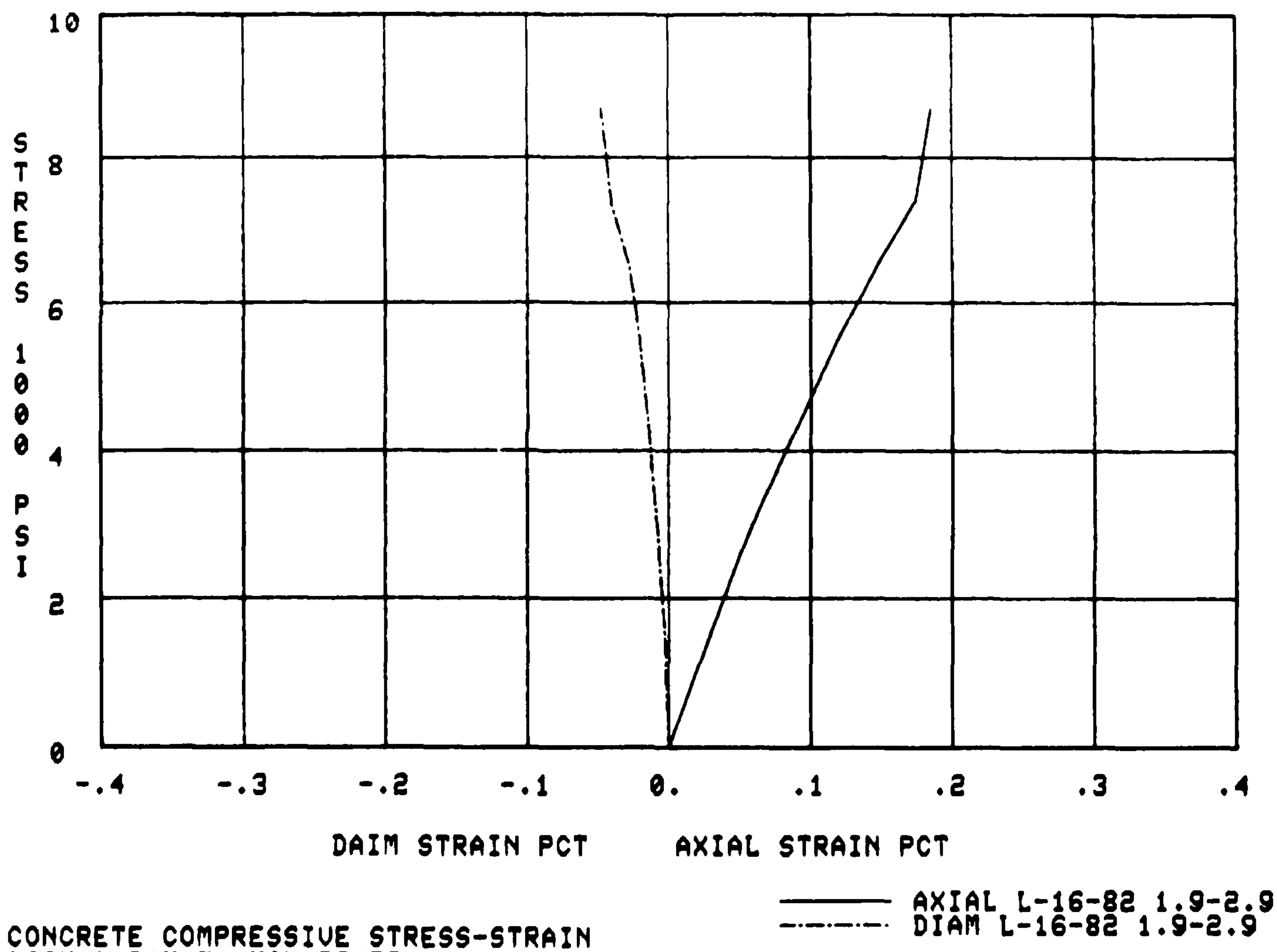

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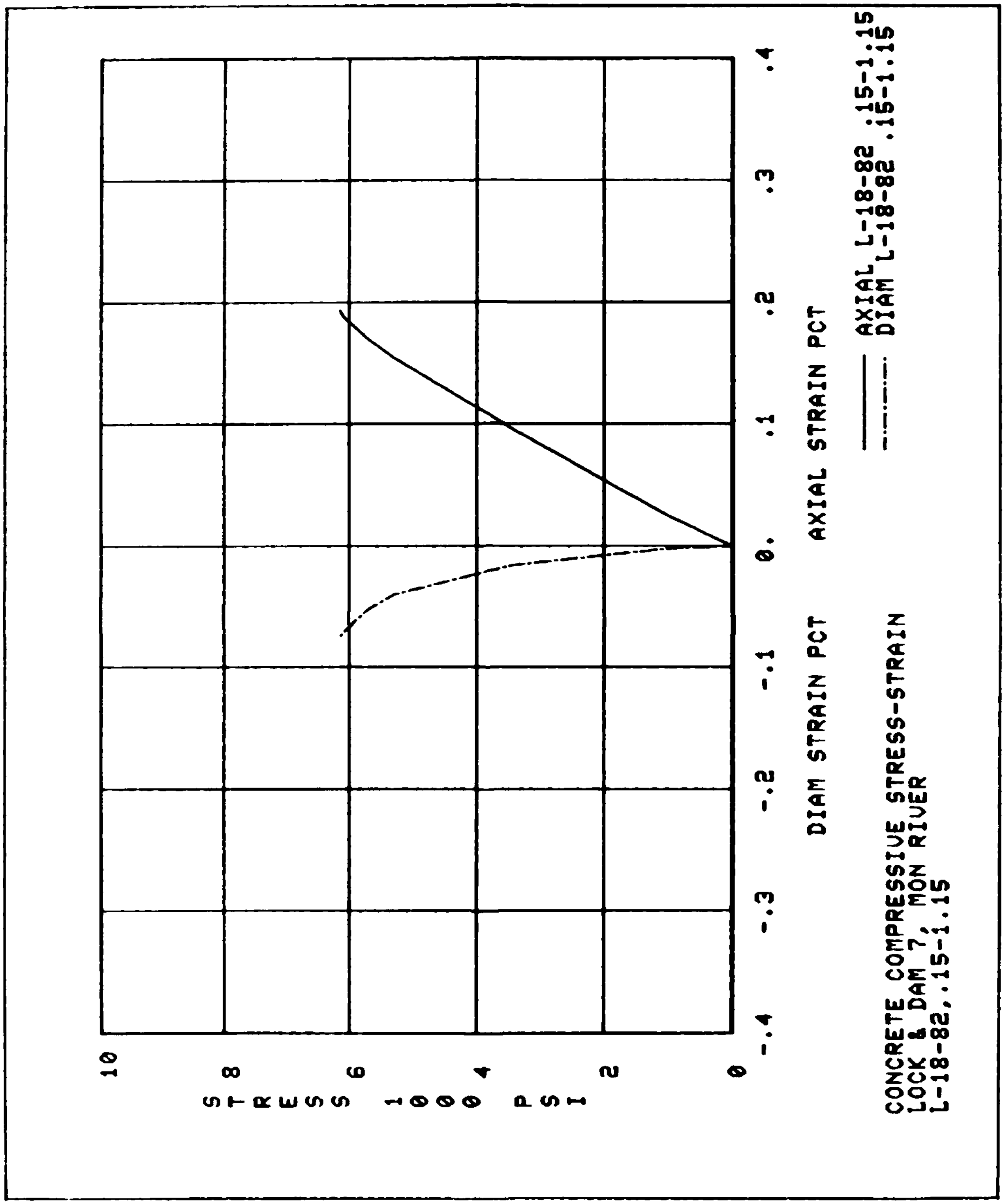
L-16-82,1.9-2.9



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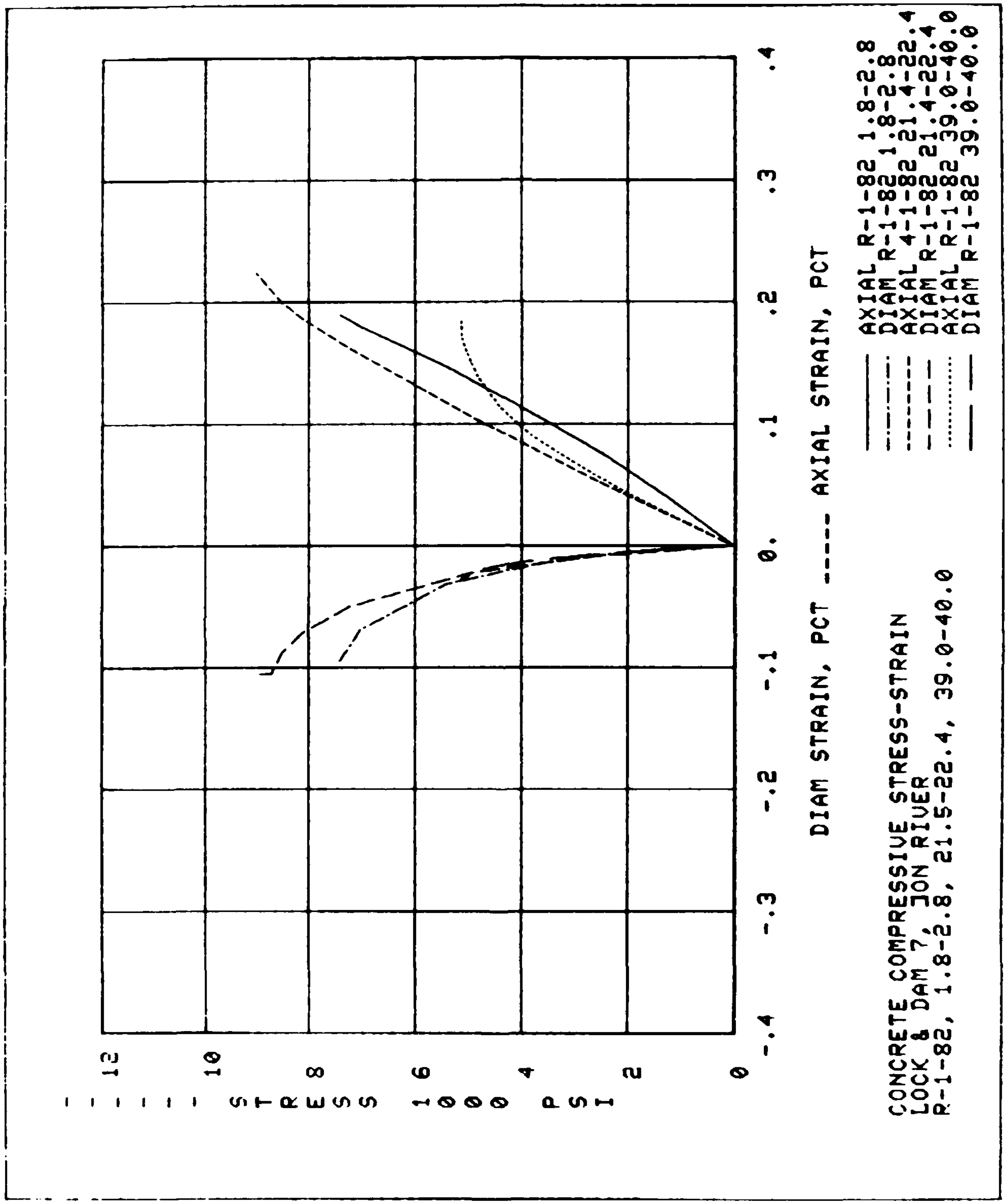
LOCK & DAM 7, MON RIVER
L-18-82,.15-1.15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LOCK & DAM 7, JON RIVER
R-1-82, 1.8-2.8, 21.5-22.4, 39.0-40.0
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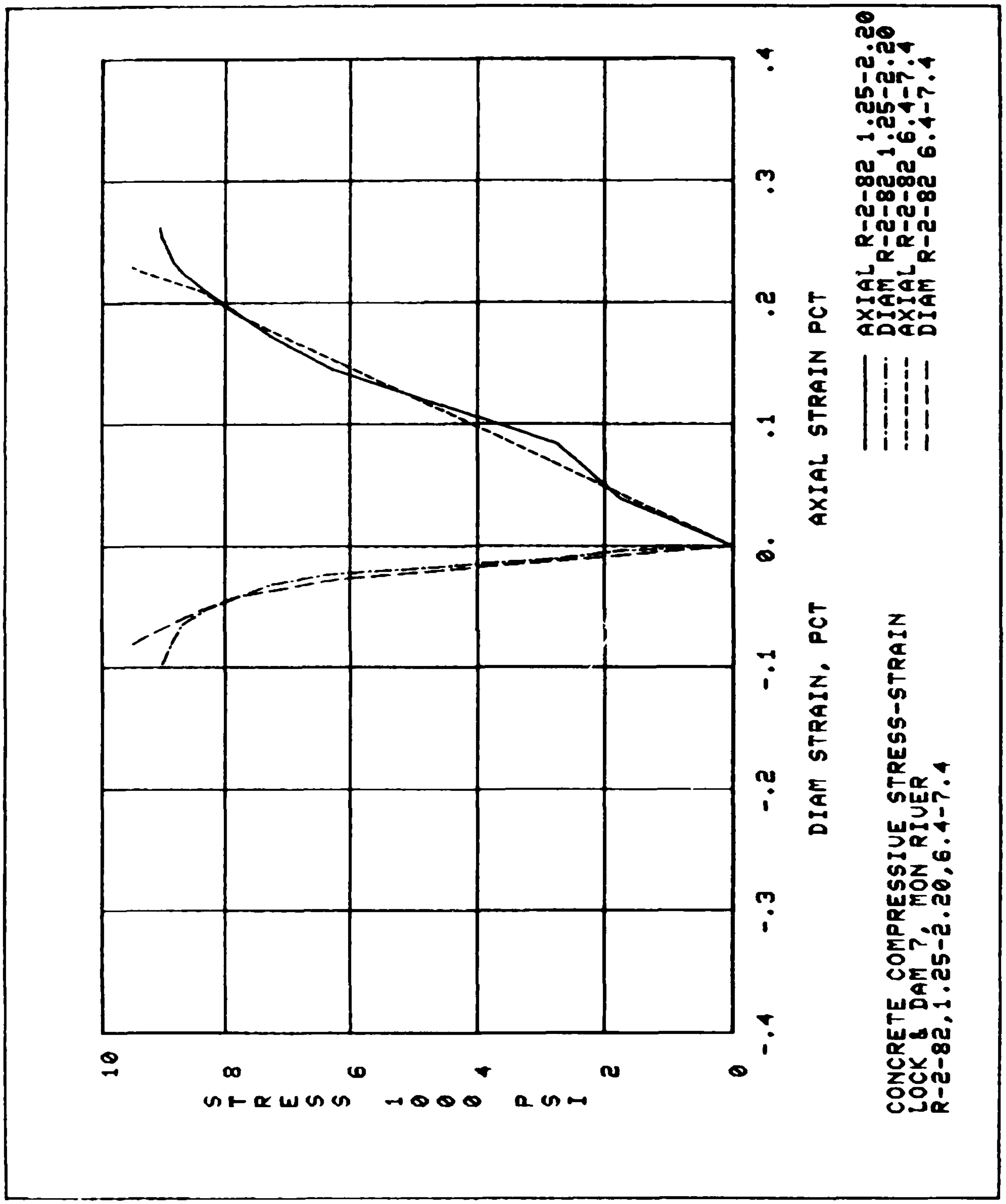
STRESS 1000 PSI
AXIAL R-1-82 1.8-2.8
DIAM R-1-82 1.8-2.8
AXIAL 4-1-82 21.4-22.4
DIAM R-1-82 21.4-22.4
AXIAL R-1-82 39.0-40.0
DIAM R-1-82 39.0-40.0



CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
R-2-82,1.25-2.20,6.4-7.4
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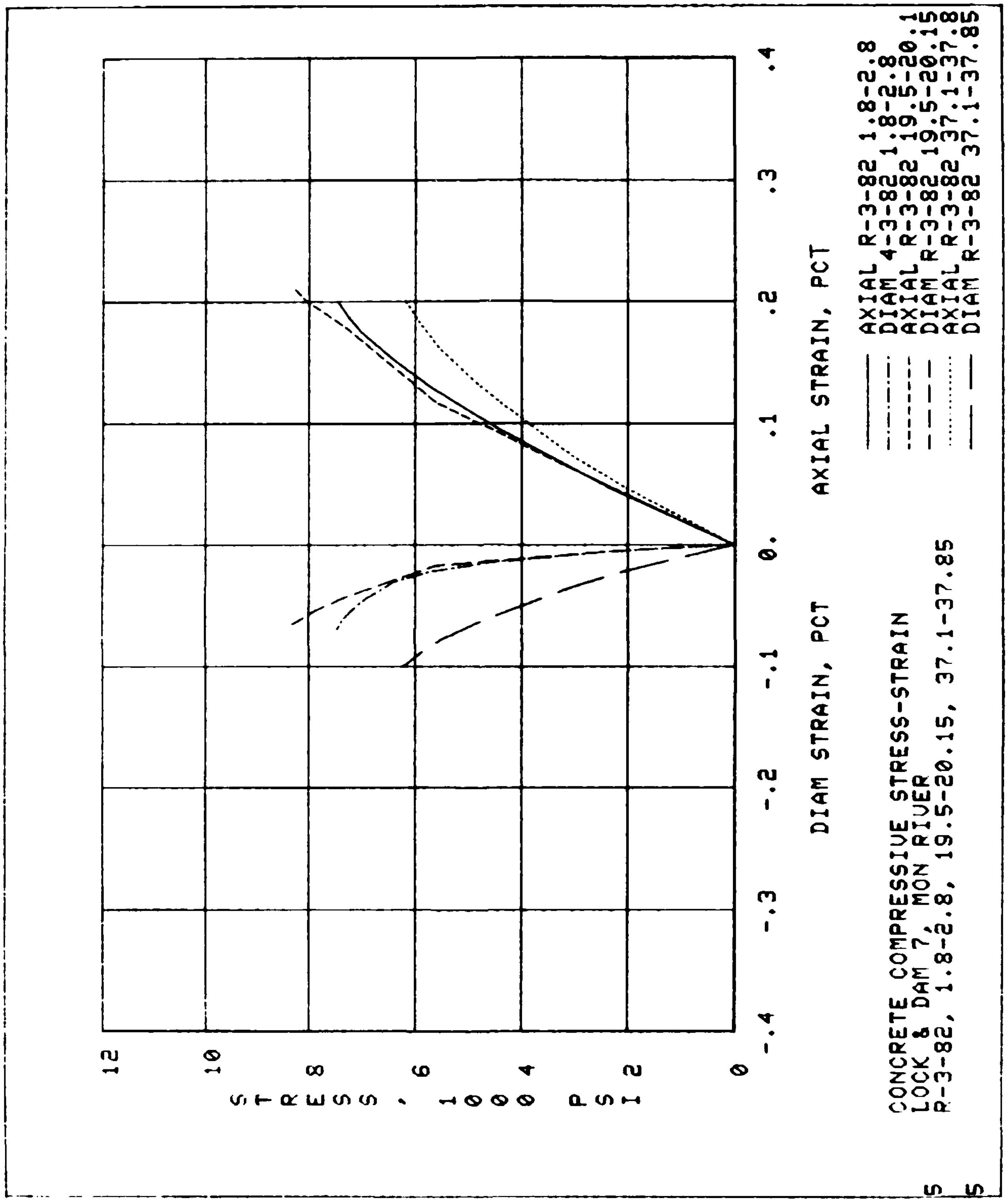
4
6
8
10
-.4
-.3
-.2
-.1
0.
.1
.2
.3
.4
DIAM STRAIN, PCT
AXIAL STRAIN PCT
AXIAL R-2-82 1.25-2.20
DIAM R-2-82 1.25-2.20
AXIAL R-2-82 6.4-7.4
DIAM R-2-82 6.4-7.4


STRESS, 1000 PSI
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6
8
10
12
-.4
-.3
-.2
-.1
0.
.1
.2
.3
.4
DIAM STRAIN, PCT
AXIAL STRAIN, PCT
AXIAL R-3-82 1.8-2.8
DIAM 4-3-82 1.8-2.8
AXIAL R-3-82 19.5-20.1
DIAM R-3-82 19.5-20.15
AXIAL R-3-82 37.1-37.8
DIAM R-3-82 37.1-37.85
CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
R-3-82, 1.8-2.8, 19.5-20.15, 37.1-37.85
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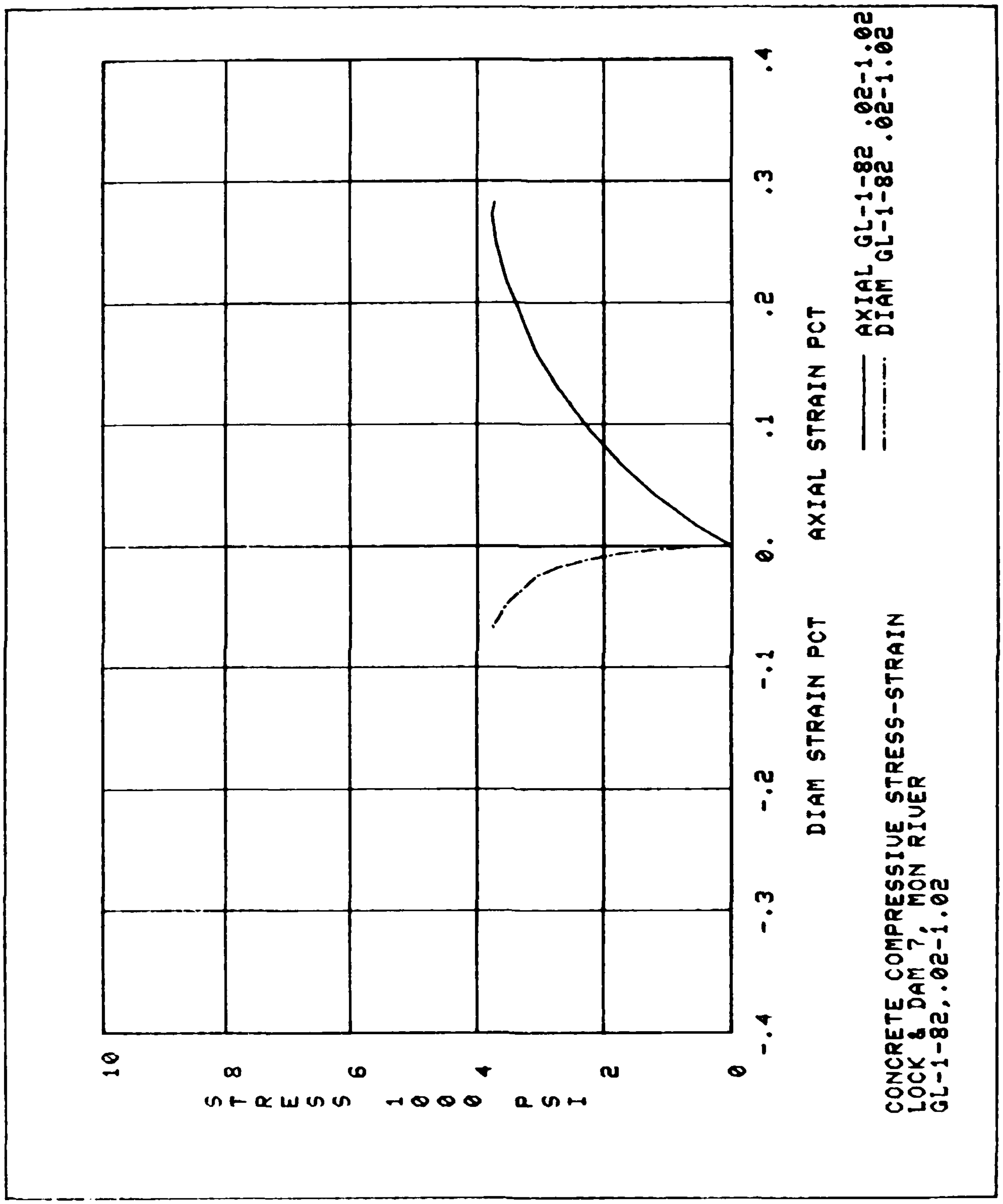
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6
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DIAM STRAIN PCT
AXIAL STRAIN PCT
AXIAL GL-1-82 .02-1.02
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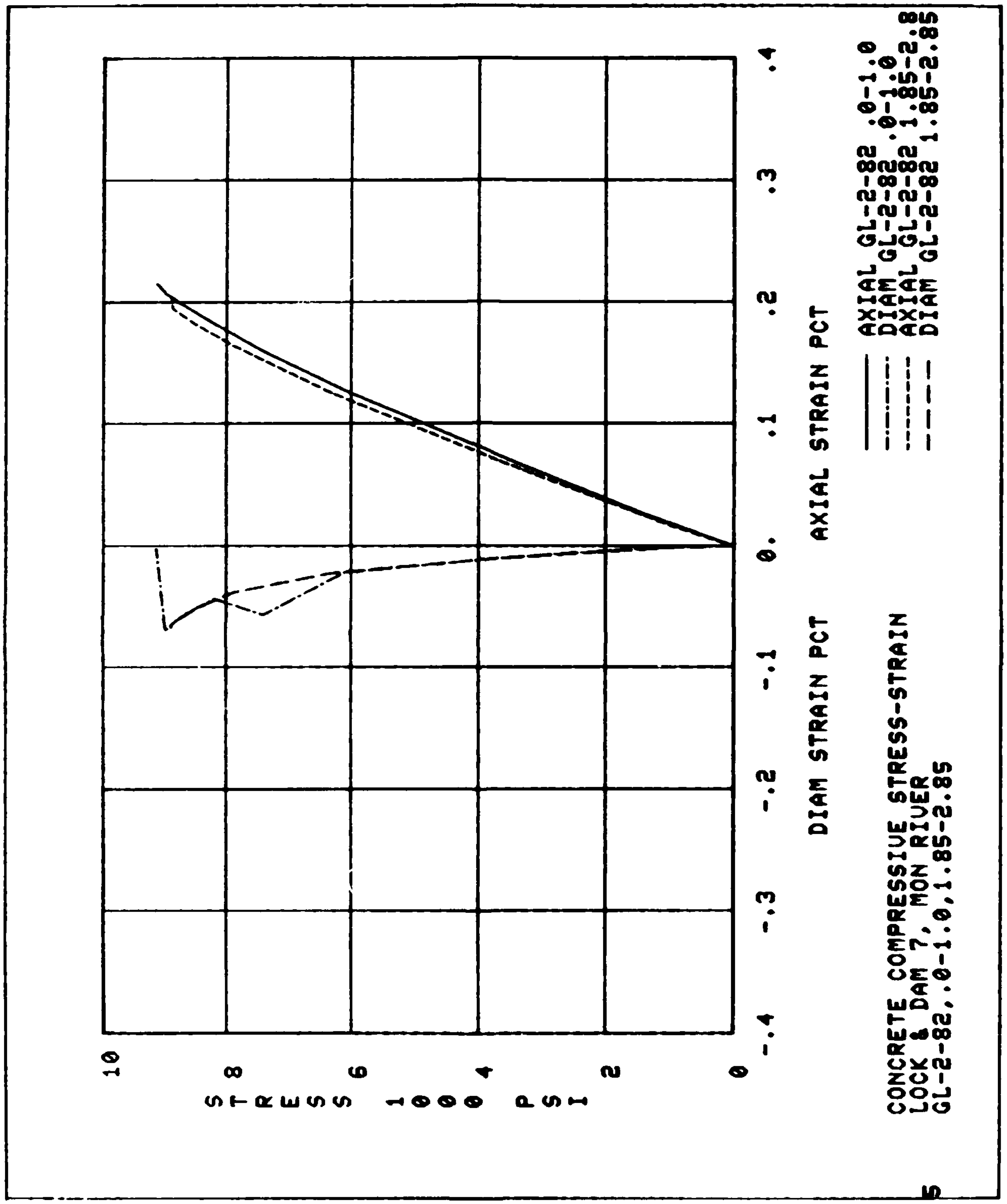
CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
GL-1-82,.02-1.02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DIAM STRAIN PCT
AXIAL STRAIN PCT
AXIAL GL-2-82 .0-1.0
DIAM GL-2-82 .0-1.0
AXIAL GL-2-82 1.85-2.8
DIAM GL-2-82 1.85-2.85
CONCRETE COMPRESSIVE STRESS-STRAIN
LOCK & DAM 7, MON RIVER
GL-2-82,.0-1.0,1.85-2.85
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END
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